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## UNIFORM MARKING OF FIRE HYDRANTS*

The American Water Works Association, recognizing that the adoption of a capacity marking scheme by any water department is optional, herewith provides the following uniform color scheme for painting hydrants rated in terms of their relative capacity.

1. Hydrants are classified as follows:—

Class A—Hydrants that on individual test usually have a flow capacity of 1,000 g.p.m. or greater.

Class B—Hydrants that on individual test usually have a flow capacity of 500 to 1,000 g.p.m.

Class C—Hydrants that on individual test usually have a flow capacity of less than 500 g.p.m.

Capacities are to be rated by flow measurements of individual hydrants at a period of ordinary demand. When initial pressures are over 40 lbs. per square inch at the hydrant under test, the rating is to be based upon 20 lbs. per square inch residual pressure, observed at the nearest hydrant, connected to the same main, from which no water is being drawn. When initial pressures are less than 40 lbs. per square inch residual pressures shall be at least half of the initial.

* This standard color scheme duplicates, in essentials, similar plans adopted by the Maine Water Utilities Association in 1929, the New England Water Works Association on March 21st, 1934, and the National Fire Protection Association on May 14th, 1936.

2. The following is the capacity-indicating color scheme:—

2.1 *Public Hydrants*

All barrels are to be painted chrome yellow except in cases where another color is desired.

Class A—Tops and nozzle caps—Green
Class B—Tops and nozzle caps—Orange
Class C—Tops and nozzle caps—Red

2.2 *Private Hydrants*

Within private enclosures, the marking is to be at the discretion of the owners.

When in public streets, they should be painted to distinguish them from public hydrants.

3. All location markers for flush hydrants should carry the same color background as stated above for class indication, with such data stencilled or painted thereon as may be deemed necessary.

4. The colors shall signify only the approximate capacity of the individual hydrant as tested alone and not its capacity when more than one hydrant in the vicinity is in use. The marking of the hydrant is not to be considered as in any way guaranteeing the capacity indicated by the color.

*Note:* This text embodies the recommendations of the Association's Committee on Uniform Marking of Fire Hydrants and is in final form. It will be presented for adoption on Monday, June 7th, 1937, at the Buffalo convention.

# COMMITTEE REPORT
# TENTATIVE SPECIFICATIONS FOR FIRE HYDRANTS*

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## Section 1—Size and Classification

1.1 The size of the hydrant shall be designated by the nominal diameter of the valve opening, which must be at least 4 inches for hydrants having two 2½ inch nozzles, 5 inches for hydrants having three 2½ inch nozzles, and 6 inches for hydrants having four 2½ inch nozzles. Hydrants shall be classed as one way, one way and steamer, two way, two way and steamer, three way, three way and steamer, four way, or double steamer, according to the arrangement of hose and steamer connection nozzles desired. The size of hydrant and the arrangement of outlets shall be as ordered by the purchaser.

1.2 The length of the hydrant shall be designated by the depth of the trench in which it is to be installed, i.e., the distance from the bottom of the connecting pipe to the ground line at the hydrant. Hydrants shall be furnished with lengths varying in multiples of 6 inches, as ordered by the purchaser.

* Proposed to supersede the "Standard Specifications for Hydrants" adopted by the American Water Works Association June 24, 1913. Submitted to the Water Works Practice Committee for Committee No. 7-E by William R. Conard, Chairman. Revised as of May 18, 1936, December 24, 1936 and January 7, 1937. Comments should be addressed to Harry E. Jordan, Secretary A. W. W. A., 29 West 39th St., New York. *There will be an open discussion of this report at the Buffalo convention on Monday morning, June 7th, 1937.*

## Section 2—Design

2.1 Hydrants may be of the compression or gate type. The design must be such that the valves can be readily removed for repairs without digging up the hydrant.

2.2 Any change in diameter of the water passage through the hydrant shall be accomplished by means of easy curves, and all outlets shall have rounded corners of good radius. The net area of the hydrant at the smallest part of the waterway, when the valve is wide open, must not be less than 120 percent of that of the valve opening.

2.3 Hydrants shall be so designed, particularly as regards the pitch of the thread of the operating stem, that, when properly operated, a water hammer will not be caused which will give an increased pressure to exceed the working pressure when such pressure is over 60 pounds, nor increase the pressure more than 60 pounds when operated under less working pressure than 60 pounds.

2.4 The design of the hydrant shall be such that in case of breakage or damage to the barrel or the operating mechanism above the ground line the valve will remain closed and sufficiently tight to avoid flooding.

2.5 When hydrant barrels are made in two or more sections, the flange or other connection near the ground line shall, unless otherwise specified by the purchaser, be at least 2 inches above the ground with the hydrant set in place.

2.6 The friction loss shall not be more than 1 lb. for each $2\frac{1}{2}$ inch hose outlet for hydrants 5 ft. or more in length discharging 250 gallons per minute for each outlet.

2.7 When so ordered by the purchaser the design shall provide lugs or other means for joint harnessing of the hydrant to the pipe lateral which runs to the street main.

2.8 The part of the hydrant known as the elbow or lateral, shall be provided with means for connecting to the supply pipe consisting either of a bell or a flange, or other type of connection as specified by the purchaser. It shall have a clear waterway of not less than 6 inch nominal diameter unless otherwise ordered. Bell or flange dimensions shall be as stated in the latest Standard Specifications for Cast Iron Pipe and Special Castings as adopted by the American Water Works Association.

2.9 The thickness of the wall of the barrel or standpipe shall be not less than the thickness of cast iron pipe of the same diameter and

for the same working pressure, and manufactured by the same process, and calculated by the same method as used in calculating the specified thickness of such pipe in the latest Standard Specifications for Cast Iron Pipe and Special Castings as adopted by the American Water Works Association. Tolerances in this thickness shall be as specified in the same specifications for Special Castings. In the cases of hydrant barrels designed to break at a particular point from impact the barrel thickness may be reduced in sufficient amount at that point to provide breakage location.

2.10 A positive operating non-corrodible drip or waste valve shall be provided and arranged so as to thoroughly drain the hydrant when the main valve is closed, and be tightly closed when the main valve is open. The seat for this waste valve, which shall be fastened in the hydrant securely, shall be made of non-corrodible material. All moving parts of the drip mechanism shall be designed so as to be easily removed without digging up the hydrant, and the waste outlet shall be arranged so that it may either discharge into the ground surrounding the hydrant or, if so specified by the purchaser, it may be tapped for piping to an adjacent sewer. The outlet from the waste valve is to be non-corrodible metal, or bushed therewith.

2.11 The operating threads of the hydrant shall be designed so as to avoid the working of any iron or steel parts against other iron or steel. Either the operating stem or the operating nut shall be of non-corrodible metal. The operating stem shall have square, V or Acme threads, and shall be sufficiently strong so that when operating the hydrant under a pressure of 150 pounds per square inch with a 15 inch wrench the factor of safety shall not be less than five (5).

2.12 Stuffing boxes may be of cast iron or bronze and shall have an internal diameter sufficient to allow at least $\frac{1}{4}$ inch width of packing space. The depth shall be at least four times the width of the packing space. Glands shall be made of solid bronze, or may be made of cast iron with bronze bushings. When a packing nut is used it shall be of bronze or other non-corrodible metal, and shall be properly secured so that it cannot turn due to operation of the hydrant. Gland bolts or studs shall be at least $\frac{1}{2}$ inch diameter if of iron or steel and $\frac{5}{8}$ inch if of bronze. The nuts shall be of bronze or suitable non-corrodible metal.

2.13 The hydrant top shall be designed so as to prevent interference with operation due to freezing. The clearance between the stem or operating nut and the hydrant top shall not be more than one

sixty fourth of an inch. The design shall be such as to make tampering difficult. Provision shall be made for convenient lubrication, to prevent both wear and corrosion, and to insure easy operation.

2.14 Nozzle caps shall be provided for all outlets. Unless otherwise ordered they shall be securely chained to the barrel with a chain made of material not less than $\frac{3}{16}$ inch in diameter, or of equivalent cross sectional area. There shall be a recess at the inner end of the threads to retain a gasket, and unless specified omitted each cap shall be provided with a suitable gasket which will provide a tight seal with the nozzle.

2.15 The direction of turning the operating nut to open the hydrant shall conform with the practice in the locality where the hydrant is to be installed. Whenever the above condition does not govern, or unless otherwise specified by the purchaser, the hydrant shall open by turning the operating nut to the left, counter clockwise. An arrow and the word—OPEN—shall be clearly cast in relief in the top of the hydrant so as to designate the direction of opening.

2.16 The parts of all hydrants of the same model and size and made by the same maker shall be interchangeable with each other without any special fitting.

2.17 All machined surfaces shall be so designated by the manufacturer on his drawings or designs. All machined surfaces shall be suitably finished for the service intended, and so that the hydrant may be readily kept free from leakage in service.

2.18 All bolt heads shall be American Standard Regular bolt heads, and all nuts shall be American Standard Heavy nuts, both according to specification B18.2-1933 of the American Standards Association, or the latest revised issue thereof. They shall be of first class workmanship and conform in physical characteristics to the specifications below listed for wrought iron or wrought steel or bronze.

## Section 3—Materials

3.1 Under these specifications, the parts to be made of cast iron shall be the elbow or lateral, the barrel or standpipe, the frost jacket if used, the hydrant head, the valve in the case of gate hydrants and the nozzle caps, together with such small miscellaneous parts where the design lends itself to the use of cast iron.

3.2 All cast iron parts shall be made from a superior quality of iron remelted in a cupola, an air or an electric furnace, producing a metal which is tough, of uniform fine grain, without chilled surfaces

and readily machined; and conforming with American Society for Testing Materials specification A126-30 class B, gray iron, or the latest revised issue thereof. All castings shall be clean and sound, without defects of any kind. No plugging, welding or repairing of defects shall be done.

3.3 All wrought iron parts shall be made from genuine wrought iron conforming with American Society for Testing Materials specification A41-30 or A84-33, or the latest revised issue thereof.

3.4 All steel parts shall be made from steel conforming with American Society for Testing Materials specification A31-24 or A107-33, or the latest revised issue thereof.

3.5 The parts to be made of bronze or other non-corrodible metal shall be the outlet nozzles, valve seat or seat ring, the drip or waste valve mechanism and other parts as elsewhere herein so specified together with such small miscellaneous parts where the design lends itself to the use of bronze.

3.6 All brass or bronze parts shall be made from a bronze alloy having a tensile strength of not less than 32,500 pounds per square inch, a yield point of not less than 16,250 pounds per square inch and an elongation of not less than fifteen percent (15%) in two inches. Any other non-corrodible alloy used shall have physical properties at least equal to this.

3.7 At least one test bar shall be made and tested by the manufacturer from each heat of metal used, in accordance with the American Society for Testing Materials specification for the material used.

3.8 The main valve of the hydrant shall be faced with a suitable yielding material such as rubber, leather or balata, except that if of the gate type a bronze ring and seat may be used.

## Section 4—Marking

4.1 All hydrants shall carry permanent markings designating the name of the manufacturer and the size of valve opening of the hydrant. All marks shall be clearly legible, located and of size so as to be readily seen after installation of the hydrant. See also Section 2.15.

## Section 5—Hose Threads

5.1 Unless otherwise specified by the purchaser the hose threads on both the hose and the steamer nozzles shall be of design and dimen-

sions according to the National (American) Standard Fire Hose Coupling Screw Thread, American Standards Association specification B26-1925.

## Section 6—Operating Nuts

6.1 The operating nut for the hydrant valve and the cap nuts on the hose and steamer nozzles shall be of design to conform with those already in service in the locality where the hydrant is to be installed. Whenever the above condition does not govern this feature the said nuts shall be of regular pentagonal shape. The dimension of this pentagon from point to flat shall be $1\frac{1}{2}$ inches at the base of the nut and $1\frac{7}{16}$ inches at the top, uniformly tapered, and the height of the nut shall be $1\frac{1}{8}$ inches. The point to flat dimension is to be measured to the theoretical point where the faces intersect without regard for any slight rounding off of the point which may actually exist. Wrenches to fit these nuts shall be made without any taper in the openings, so as to be readily reversible.

## Section 7—Testing and Inspection

7.1 Hydrants for a working pressure of 150 pounds per square inch or less shall be tested after assembly by means of hydraulic pressure of 300 pounds per square inch. The test shall be made with pressure throughout the interior of the hydrant and shall also be made with the hydrant valve closed and the pressure on the inlet side. There shall be no leakage or evidence of imperfection in either test.

7.2 Representatives, such as the engineer or inspecting engineer of the purchaser, shall be at liberty at all times to inspect materials being made for his or their principals, and shall have access to all parts of the works where such material is being produced, and to approve the material as to quality and workmanship and to disapprove that which in his judgment does not conform to the requirements of the purchaser or the spirit and intent of these specifications.

## Section 8—Painting

8.1 All iron parts of the hydrant, inside and outside, except the outside surfaces above the ground line, shall be thoroughly cleaned and then painted with or dipped in an asphaltic or bituminous base paint, unless otherwise specified. If these parts are painted they

shall be painted with two coats, the first being allowed to dry thoroughly before the second is applied. If these parts are dipped they shall be preheated and the dip shall be hot.

8.2 The outside of the hydrant above the ground line shall be thoroughly cleaned and then painted with two coats of paint of a durable and weather proof composition, and of a composition that can later be satisfactorily painted over with a linseed oil or other suitable paint if so desired. The color or colors to be used, shall be as specified.

# ERRORS IN PRESSURE READINGS DUE TO WRONG GAUGE CONNECTIONS*

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In order to determine the pressure in a pipe, the usual practice is to drill a hole in and normal to the wall of the pipe and then to tap this hole for a pipe connection, the entire work being done from outside. A $\frac{1}{4}$-inch or $\frac{3}{8}$-inch connection is very common, in which case the hole is drilled full size for the tap through the pipe, while the pressure gauge pipe is only screwed part way. In very many cases, the inside of the pipe is inaccessible and it is, therefore, impossible to ascertain the conditions there or to find out what lumps or rough spots are in the vicinity of the drilled hole and, in general, very little attention is paid to the condition of the inside of the pipe.

Where centrifugal pumps and some other machines are supplied, the customer is frequently saved some trouble in connection with the drilling of the holes by a practice among some manufacturers of tapping holes in the suction and discharge flanges, as shown in Figure 1, these holes being suitable for the connection of the gauges. In this case, the hole is often situated within three-quarters of an inch of the face of the flange and frequently there is no way of finding out how the opening in the flange registers with that in the pipe connection which is bolted to it.

The cores in the various pipes are frequently not exactly central and often the two halves of the core do not exactly match and, in addition, they are often not of the exact size specified. Since the workman in connecting up two pipes, or a pipe to a pump, is careful to see that the outsides of the flanges exactly coincide and pays no attention to whether the openings inside match or not, the result is that frequently the two openings at the flanges are not concentric, and even when they are, one of them is often larger than the other, and neither of them may be exactly round.

* Contributed record of research. Revised form of paper read before Canadian Section.

It is, further, the custom to drill the hole for the gauge connection on a diameter which is normal to the plane of a bend, if such bend exists in the pipe, because the general impression is that if the hole is made in the plane of the bend the gauge may read too high if on the outside of the latter, or too low if on the inside of it.

The author has, for many years, been conducting acceptance tests on centrifugal pumps and other machinery, and is of the opinion that errors are frequently made in pressure measurements due to the way in which the gauges are attached. Projections, in the cases already enumerated, where they occur upstream or downstream from the pressure opening, were thought to cause incorrect pressure readings, and the experiments described in this paper proved that to be so.

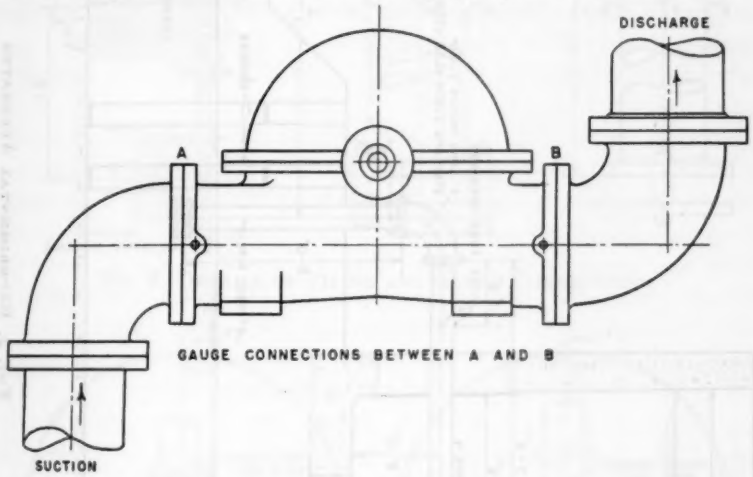


FIG. 1. SHOWING POSITIONS OF GAUGE CONNECTIONS OFTEN USED ON PUMPS

## EXPERIMENTAL STUDIES

In order to determine the effect of projections or lumps in the pipe close to the gauge opening, the experiments described in this paper were carried out. The size pipe selected for the experiments was 6 inches, as this was thought to be amply large to show the effects on a good scale, and experiments were made with a pipe line consisting of one straight stretch of 7 ft. 6 in. long connected by a 90 degree elbow to a second stretch 5 ft. long. Two types of elbow were used, both being flanged cast fittings of standard weight, one of these being of short radius and the other long radius, but these were taken from the manufacturer's stock at random. The longer pipe, Figure

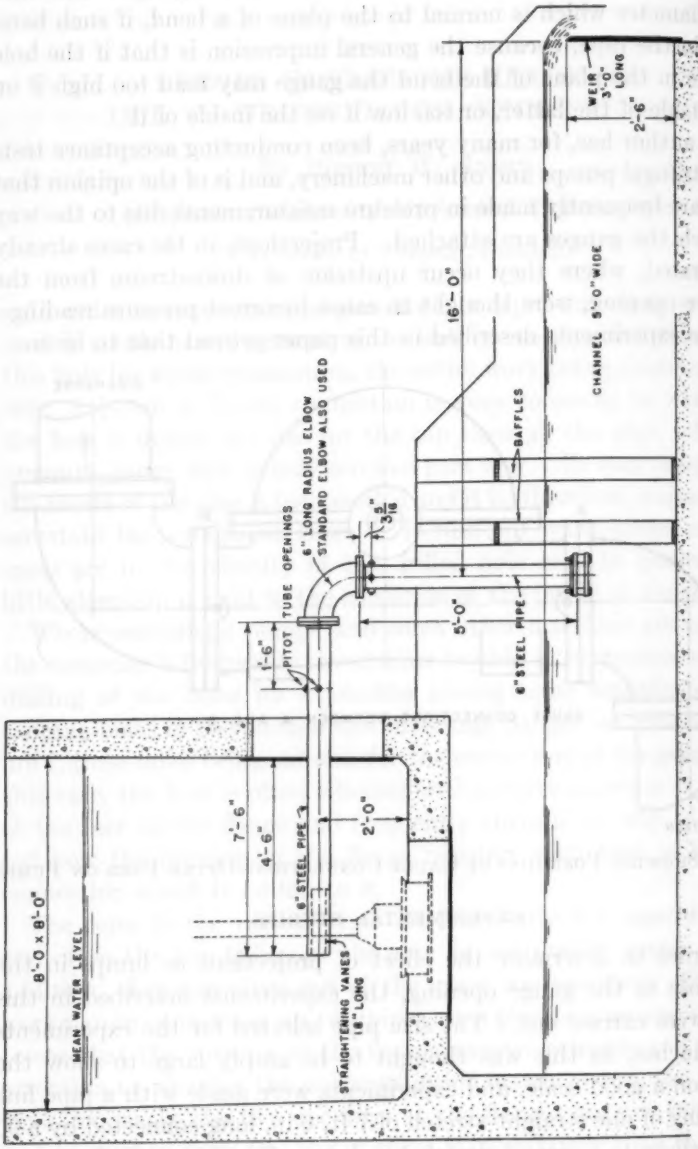


FIG. 2. EXPERIMENTAL APPARATUS

2, was inserted through the side of a tank which had a horizontal cross sectional area of 64 sq. ft. and a spillway at the top by which it

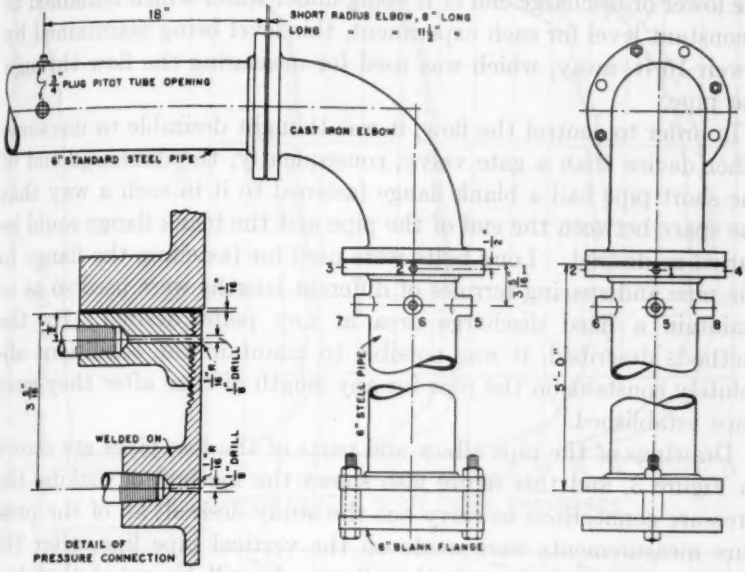


FIG. 3. DETAILS OF PIPING AND GAUGE CONNECTIONS

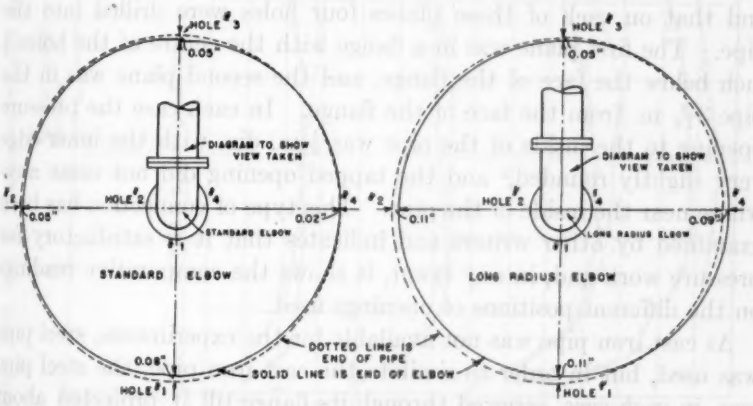


FIG. 4. POSITIONS OF OPENINGS IN ELBOW AND PIPE, AT THE TOP FLANGE OF THE VERTICAL PIPE

was possible to maintain a perfectly constant level. Straightening vanes 18 in. long were put at the entrance end of this pipe so as to guide the water as nearly parallel to the axis as possible, although

these vanes were not necessary. The shorter length of pipe was connected to the elbow in such a way as to stand in a vertical position, the lower or discharge end of it being under water which remained at a constant level for each experiment, this level being maintained by a weir 16 ft. away, which was used for measuring the flow through the pipe.

In order to control the flow, it was thought desirable to use some other device than a gate valve; consequently, the discharge end of the short pipe had a blank flange fastened to it in such a way that the space between the end of the pipe and the blank flange could be varied as desired. Long bolts were used for fastening the flange to the pipe and spacing ferrules of different lengths were used so as to maintain a fixed discharge area at any point desired. By the methods described, it was possible to maintain the conditions absolutely constant on the pipe for any length of time after they were once established.

Drawings of the pipe elbow and parts of the two pipes are shown in Figure 3, and this figure also shows the method of making the pressure connections to carry out the study desired; all of the pressure measurements were made on the vertical pipe line, after the water had passed through the elbow. It will be noted that two planes were selected on which the pressure connections were placed, and that on each of these planes four holes were drilled into the pipe. The first plane was in a flange with the centre of the holes $\frac{1}{2}$ inch below the face of the flange, and the second plane was in the pipe $3\frac{5}{16}$ in. from the face of the flange. In each case the pressure opening to the inside of the pipe was $\frac{1}{8}$ in. dia. with the inner edge very slightly rounded,[1] and the tapped opening did not come anywhere near the inside of the pipe. This type of connection has been examined by other writers and indicates that it is satisfactory for pressure work and, in any event, it shows the comparative readings on the different positions of openings used.

As cast iron pipe was not available for the experiments, steel pipe was used, but in order to similate the cast iron pipe, the steel pipe was, in each case, screwed through its flange till it projected about $\frac{1}{8}$ in. beyond it, and then the whole pipe was put in the lathe and the entire face of the flange and end of the pipe were faced at the same

[1] See paper by Allen and Hooper, Vol. 54, American Society of Mechanical Engineers, Hydraulic Division.

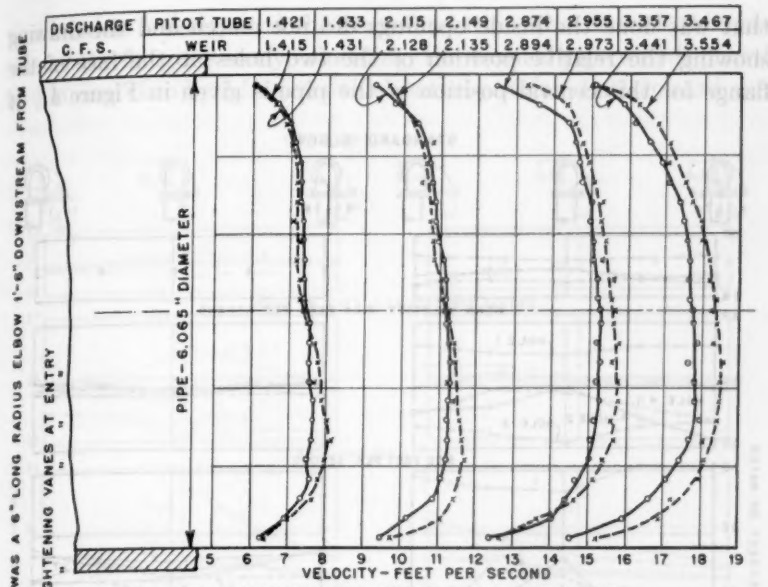


FIGURE 5. HORIZONTAL PITOT TUBE TRAVERSES

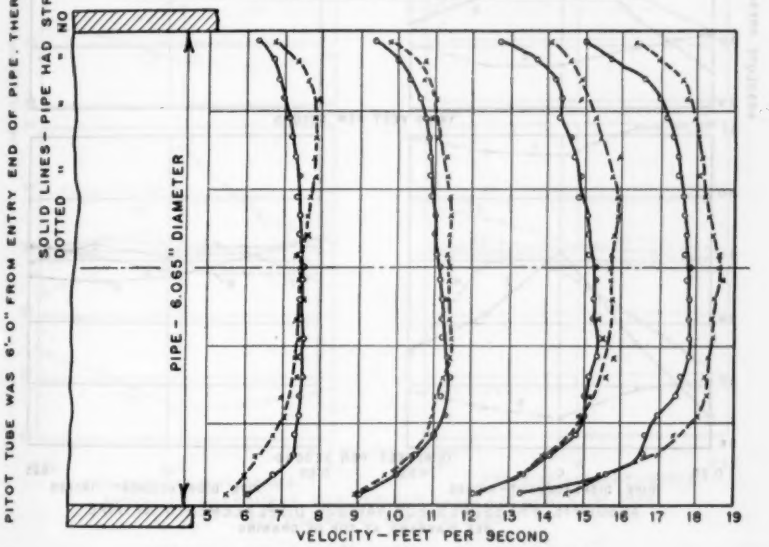
PITOT TUBE WAS 6'-0" FROM ENTRY END OF PIPE. THERE WAS A 6" LONG RADIUS ELBOW 1'-6" DOWNSTREAM FROM TUBE
SOLID LINES - PIPE HAD STRAIGHTENING VANES AT ENTRY
DOTTED " " " NO " "
PIPE - 6.065" DIAMETER
5 6 7 8 9 10 11 12 13 14 15 16 17 18 19
VELOCITY - FEET PER SECOND

FIGURE 6. VERTICAL PITOT TUBE TRAVERSES

time, which gave as smooth an end to the pipe as one would find in cast iron.

In bolting the pipes together in the first instance, the outside of the flanges were made to match just as nearly as possible, but when

that was done the inside openings did not match and the drawing showing the relative position of the two holes at the face of the flange for this central position of the pipe is given in Figure 4. It

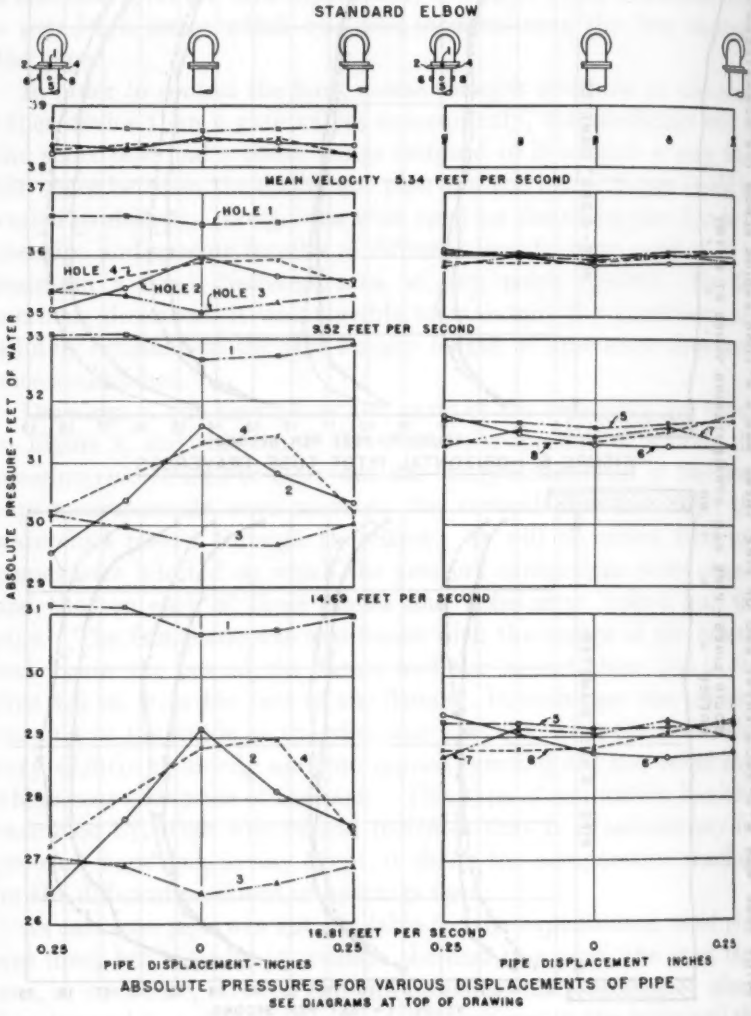


FIG. 7

shows that there was a variation of 0.11 in. at some points between the two openings and that they were not round and were far from coinciding at most places. After making an experiment with the

pipe in what is designated the central position, that is with the outsides of the flanges exactly coinciding, the vertical pipe was displaced by $\frac{1}{8}$ in. and then by $\frac{1}{4}$ in. from this central position, both inward and

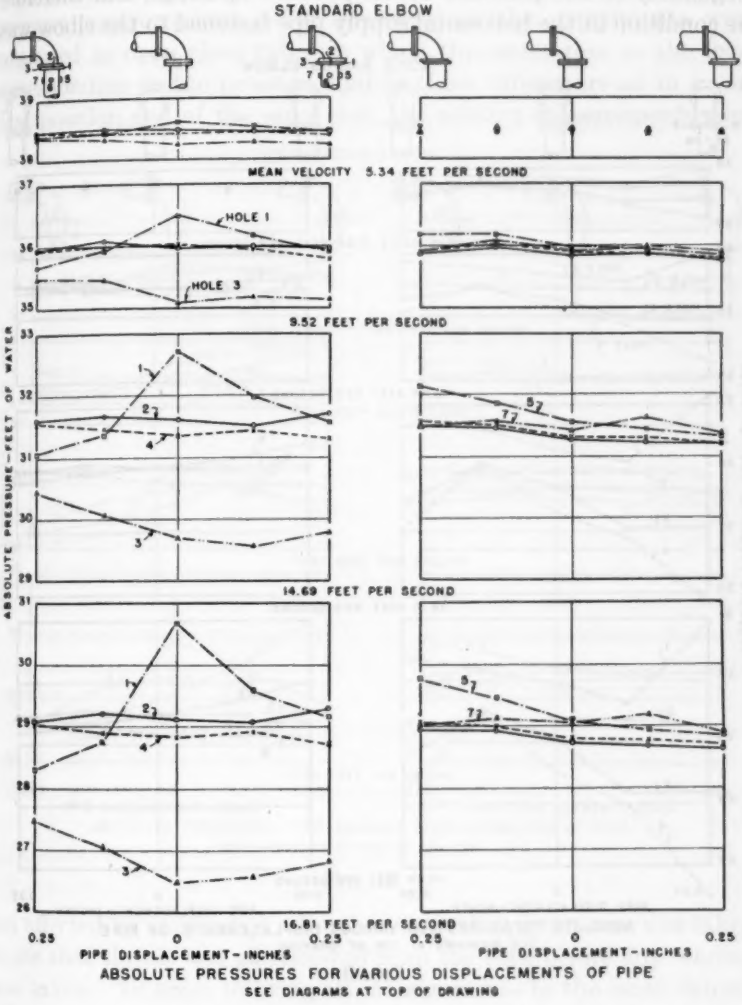


FIG. 8

outward and also sideways in both directions, complete pressure readings being taken for each setting, so that the maximum effect corresponded to a possible displacement of $\frac{1}{4}$ in. in any direction,

resulting in displacements of the openings at the flange up to 0.25 + 0.11 = 0.36 in. in some cases.

In making the experiments, it was desired to confine all of the irregularity to the positions of the pressure openings, and therefore the condition in the horizontal supply pipe fastened to the elbow was

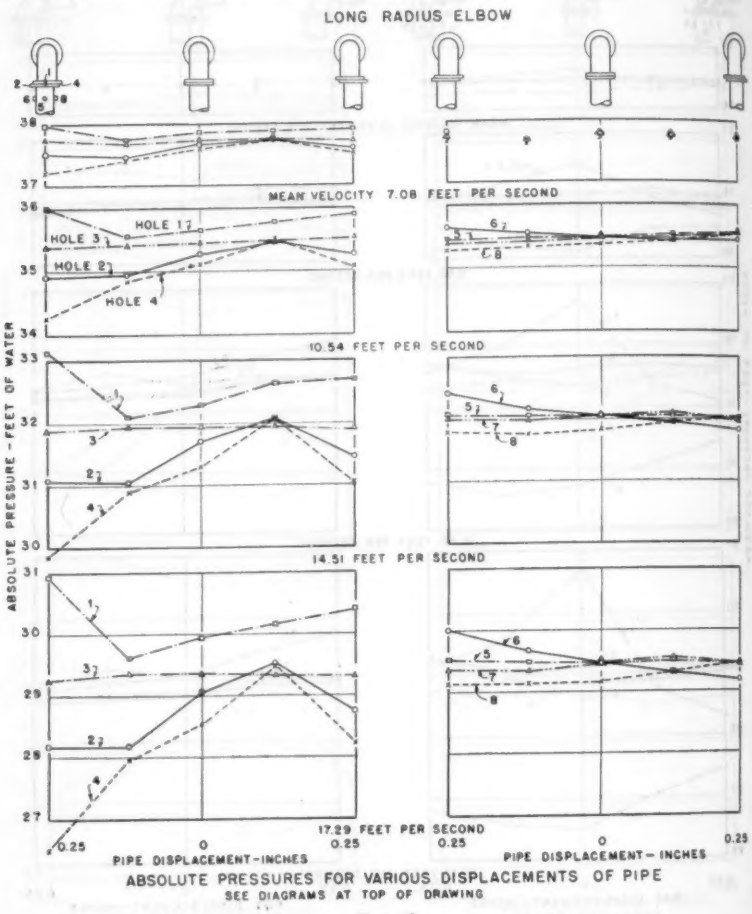
LONG RADIUS ELBOW
MEAN VELOCITY 7.08 FEET PER SECOND
HOLE 1
HOLE 3
HOLE 2
HOLE 4
10.54 FEET PER SECOND
14.51 FEET PER SECOND
17.29 FEET PER SECOND
ABSOLUTE PRESSURE - FEET OF WATER
PIPE DISPLACEMENT - INCHES
PIPE DISPLACEMENT - INCHES
ABSOLUTE PRESSURES FOR VARIOUS DISPLACEMENTS OF PIPE
SEE DIAGRAMS AT TOP OF DRAWING

FIG. 9

examined by pitot tube, this tube being inserted in the pipe 18 inches from the face of the elbow. Traverses were then made, both on the horizontal and vertical diameters of the pipe, and the results of these traverses are shown on Figures 5 and 6. They indicate a remarkably good distribution of the water in the supply pipe, at least as

good as one would expect in any case occurring in practice. But after these traverses had been made the pitot tubes were removed, and the holes through which they had been passed were blocked up so as to leave the pipe perfectly smooth on the inside before making pressure measurements. The pressures in the experiments were measured in open glass tubes in which the water rose to the level corresponding to the pressure, and as these tubes were all in a vertical position and of the same size, the relative measurements were

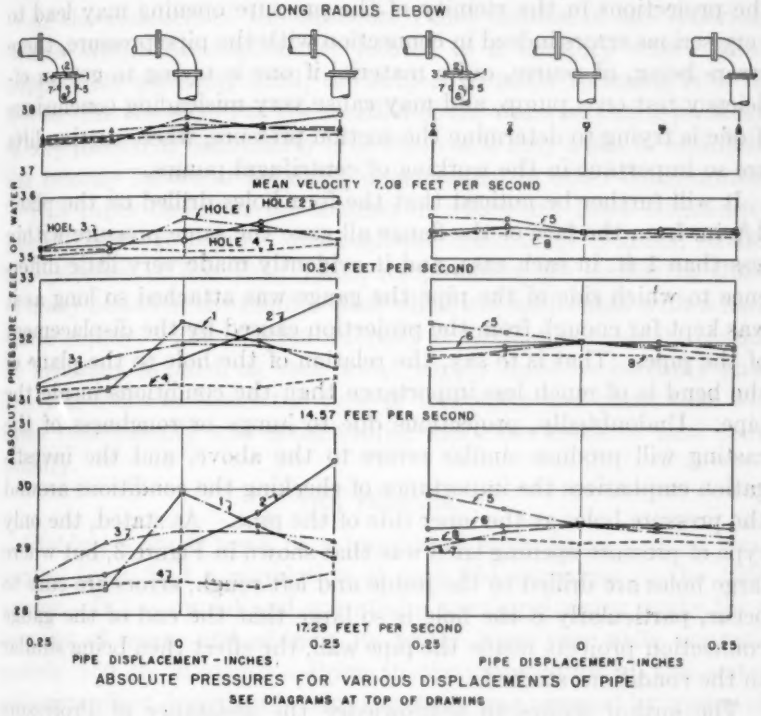
LONG RADIUS ELBOW
MEAN VELOCITY 7.08 FEET PER SECOND
HOLE 1
HOLE 2
HOLE 3
HOLE 4
10.54 FEET PER SECOND
14.57 FEET PER SECOND
17.29 FEET PER SECOND
ABSOLUTE PRESSURE - FEET OF WATER
PIPE DISPLACEMENT - INCHES.
PIPE DISPLACEMENT - INCHES
ABSOLUTE PRESSURES FOR VARIOUS DISPLACEMENTS OF PIPE
SEE DIAGRAMS AT TOP OF DRAWING

FIG. 10

not affected by capillarity. Further than that, great care was taken to see that the air was all removed from the tube before any reading was taken. In order to bring all measurements to the same datum, they have all been referred to the centre line of the horizontal pipe.

## DISCUSSION OF RESULTS

The results of the measurements are shown on Figures 7, 8, 9 and and 10, the experiments having been run at velocities varying from

5.34 to 17.29 ft. per sec., four velocities being taken with each type of elbow. These results are very interesting and show that, with the set of holes drilled in the flange, variations exceeding 4 ft. occur between measurements made at different holes and, curiously enough, the greatest difference is not between two holes in the plane of the bend, but sometimes between two holes at 90 degrees from one another, as is shown in the lower diagrams of Figures 7 and 9. The diagrams themselves are worthy of quite careful study and show that the projections in the vicinity of the pressure opening may lead to very serious errors indeed in connection with the pipe pressure, these errors being, of course, quite material if one is trying to get an efficiency test on a pump, and may cause very misleading conclusions if one is trying to determine the suction pressure, where suction lifts are so important in the working of centrifugal pumps.

It will further be noticed that the four holes drilled on the plane $3\frac{5}{16}$ in. from the face of the flange all gave the same pressure within less than 1 ft. in each case, and it evidently made very little difference to which side of the pipe the gauge was attached so long as it was kept far enough from the projection caused by the displacement of the pipe. That is to say, the relation of the hole to the plane of the bend is of much less importance than the conditions inside the pipe. Undoubtedly, projections due to lumps or roughness of the casting will produce similar errors to the above, and the investigation emphasizes the importance of checking the conditions around the pressure holes at the inner side of the pipe. As stated, the only type of pressure opening tried was that shown in Figure 3, but where large holes are drilled to the inside and left rough, errors are sure to occur, particularly if the hole is so large that the end of the gauge connection projects inside the pipe wall, the effect then being similar to the conditions studied.

The author wishes to acknowledge the assistance of Professors Taylor and Heard, who took most of the readings on which the results are based.

# WATER SOFTENING AT GLIDDEN, IOWA*

By C. Maxwell Stanley

(*Young & Stanley, Inc., Engineers, Muscatine, Ia.*)

As the members of the Missouri Valley Section of the American Water Works Association are located in a territory where water supplies are generally hard, the subject of water softening is of interest. So, the record of the solution of the water softening problems of any particular community is of value as it may assist in establishing priority for water softening development in other communities.

This paper describes the manner in which one Iowa community has solved its problem. Although the community is small, having a population of less than 1,000, its problems are similar to those of many others in the middle West which are confronted with a hard water supply. The solution adopted at Glidden necessarily cannot be generally applied, as each water supply must be carefully analyzed in order to select the type of treatment and plant that will most economically and satisfactorily meet the needs of that community.

## THE PROBLEM

Glidden, a small town in a prosperous agricultural area of west central Iowa, had a population of only 854 according to the 1930 census. It has owned and operated a municipal water works for a number of years and obtains its water from two wells approximately 150 ft. in depth. Prior to the installation of the water softening plant described herein, the water was pumped by means of turbine-type pumps into an elevated tank and from there into the mains of the town. The total consumption of water averages only 30,000 gallons per day, or around 11,000,000 gallons per year. To those accustomed to dealing with larger plants, these figures are so small that water softening might appear neither desirable nor economical. However, the officials of Glidden did not feel so. Their only practical source, wells, produced water that was very hard, and entirely unsuited for washing purposes without the use of large

*Presented before the Missouri Valley Section, October, 1936.

quantities of soap and water softening compounds. It was furthermore objectionable for use in boilers or other heating apparatus, and had all of the other undesirable features usually accompanying a hard water. In addition, there were continuous complaints of "red water" which was generally found over the town, and which had been attributed to a high iron content in the water.

The town officials requested the writer's firm to investigate the methods, costs, and feasibility of eliminating the hardness from the water and of eliminating the "red water."

An analysis of the water supply indicated that the total hardness was slightly over 400 parts per million, or approximately 24 grains per gallon and that practically all of this hardness was carbonate. It indicated further that the iron content in the water as it came from the wells was only 0.4 parts per million, but that the pH (hydrogen-ion concentration) of this water was only 7.0.

The pH of 7.0 indicated that the water from the wells was on the border line between an acid and alkaline condition. Such a water dissolves minute quantities of iron from the mains, and thus is able to create a "red water" condition even though the iron content of the water is low.

Thus the problem of improving the water supply was a two-fold one: first, that of reducing the hardness of the water by appropriate treatment; and, second, that of eliminating the "red water" by stopping the iron pick-up in the mains resulting from the aggressive condition of the water. The first part of the problem was solved by the installation of water softening equipment, and the second part by the installation of a chemical feeder to add alkali to the water, thus eliminating the acid condition.

## ALTERNATE TYPES OF PLANT

With the general problem defined, it was possible to make comparative analyses of the different types of equipment and treatment that might be used.

Inasmuch as the hardness was almost entirely carbonate, the use of excess lime and lime and soda ash treatment was naturally considered. The use of zeolite equipment was also analyzed. After making comparative estimated construction and operating costs, and after studying the advantages and disadvantages of the two treatments, an automatic zeolite plant was recommended. The primary reasons for this selection are as follows:

1. *Construction Cost.* In making estimates of the cost of construc-

tion of the two types of plants, it was necessary to consider different sites. An automatic zeolite plant could be located in an extension to the existing pumping plant and operated satisfactorily as only a small amount of attendance is required. However, with the lime process, it was deemed desirable to locate the treatment plant several blocks from the pump house and adjacent to the municipal electric plant, in order to economize by allowing the electric plant operators to look after the water softening plant. This would have involved the purchase of additional property and the construction of additional mains, thus increasing the cost of the lime treatment plant. Under these conditions, the estimated cost of the automatic zeolite plant was $9,500 as compared to $13,800 for a plant of the conventional type. Had it been feasible to locate the lime plant adjacent to the water works, the estimated cost would have been only $10,800, but additional operating labor would have been necessary.

2. *Operating Costs.* A comparison of the operating costs of the two plants showed an estimated cost, based on the consumption of approximately 11,000,000 gallons per year, as follows:

| | |
|---|---|
| Zeolite Treatment | $820 |
| Lime Treatment | $1,030 |

These costs cover maintenance, chemicals, power, etc., but do not in either case include any labor.

3. *Sludge Disposal.* The use of a lime plant would have carried with it the problem of disposal of sludge. The sites available for the location of the plant did not afford any practical method of disposal of this sludge, which would have created a troublesome and possibly expensive situation. The use of the zeolite plant completely eliminated the problem of sludge disposal.

4. *Simplicity.* For the size of the plant considered, it was felt that the zeolite plant offered a simpler process than the lime treatment and that it could be more effectively operated by unskilled personnel than would have been the case with the lime process.

5. *Further Treatment.* There was no indication that the further treatment which would have been available with the filtration in the conventional process was necessary and, hence, in this instance there was no advantage resulting from this further treatment.

## DESIGN OF PLANT

The general arrangement and layout of the plant is very simple. It involved the construction of an extension approximately 14 ft. x

15 ft. on the existing pump house. This addition was of concrete block with a wood roof and concrete floor.

The zeolite equipment consists of two Permutit Company units, each 42 inches in diameter and 10′-6″ high and each having a capacity of 50 gpm., which closely matches the capacity of the turbine pumps. The units are provided with automatic control so arranged that each unit, after softening 18,000 gallons of water, automatically backwashes, fills with brine, and rinses. The piping is arranged in such a manner that the turbine pumps force the water directly from the wells through the softening units and into the mains. Water for backwashing and rinsing the softener is obtained from the elevated tank.

An automatic proportioner is provided so that a pre-determined amount of untreated water can be by-passed around the softeners and the hardness of the effluent thus adjusted.

Outside of the building, an 18 ft. x 27 ft. x 9 ft. underground concrete storage tank was constructed, having a capacity of approximately 50 tons of salt. The salt is flooded with water and the brine is taken out of a sump by an automatic brine pump and forced into a measuring tank from whence it is injected into the softeners at time of regeneration.

An automatic chemical feeder is provided for feeding an alkali into the water in order to adjust the pH of the effluent.

The project was placed in operation in June, 1936. The total cost of the improvement, including all overhead items, was slightly under the estimated cost of $9,500. This was financed partially by money on hand in the Water Works Fund and the balance was covered by the issuance of $7,500 of Revenue Bonds, maturing over a 10-year period.

## OPERATION OF PLANT

The operation of the plant has proved to be very simple. A town employee who looks after the water works system, as well as the streets, visits the plant for a few minutes each day. At that time, he does the necessary janitor work and fills the chemical feeder. The pumps are remotely controlled from the municipal electric plant and the water softening process is entirely automatic.

Since the plant has been in operation, the daily pumpage has averaged about 37,800 gallons per day. This is above the average for the year due to the higher summer consumption. Such opera-

tion means that each of the units regenerate about once a day. During this time, the plant has been operated practically all of the time with the automatic proportioner completely closed so that zero hardness water is produced. Salt consumption has been approximately 0.32 lbs. per grain of hardness per 1,000 gallons of water treated.

The solution tank of the chemical feeder is filled about six times a week. Nine pounds of sodium silicate and twelve pounds of caustic soda are added at each filling. The combined addition is approximately 50 parts per million and this is sufficient to raise the pH of the water from 7.0 to between 7.8 and 8.0 and satisfactorily overcomes the tendency of the water to pick up iron from the mains. The alkalinity of the water is tested by means of a phenolphthalein indicator.

To date, the cost of operation has been approximately six cents per 1,000 gallons of water treated, which is substantially under the estimated cost.

## PUBLIC ATTITUDE

A check of the attitude of the water users of the town towards the water softening plant indicates that they are well satisfied with the results that have been obtained. All of those interviewed state that the soap consumption has been materially decreased. Many of them state that they are still using more soap than is required because they have not become accustomed to using so little soap.

Soon after the plant started operation there were some complaints as to the taste of the water which was traced to the improper proportioning of the alkali added to the water, due to a misunderstanding on the part of the operator. When this condition was remedied, the complaints were removed.

## ECONOMICS

It is, of course, impossible to make a definite determination as to the savings which have resulted from the installation of this plant. These savings include a number of items, some of which are as follows:

1. Saving in soap and other water softening preparations.
2. Elimination of extra expense of cisterns and double plumbing systems.
3. Saving on fabrics.

4. Saving in heat losses in hot water heaters and boilers.
5. Saving in expense of operating steam and hot water systems.

Except for the first of these items, soap, it is impossible to make any sort of a definite estimate as to the amount of the savings. In a recent paper by the writer ("Economics of Water Softening," C. Maxwell Stanley, Jour. A. W. W. A., Vol. 28, p. 469) the soap savings in a community the size of Glidden, for a reduction of 400 parts per million, were estimated at approximately $2,000 per annum.

The total annual costs of operating this plant with present consumption will not exceed $800 per year, plus fixed charges on the investment, which at 10 per cent will amount to $950 per year, making a total annual cost of $1,750. This estimated annual cost is less than the probable savings from soap alone, entirely neglecting the other items of savings and comfort which result from the use of a soft water.

In conclusion, it may be said that the Town of Glidden, Iowa, has found a solution to its problem of an unsatisfactory hard water and while their solution may not be applicable to other communities, it at least indicates that water softening plants can be installed and operated in our smaller communities and that they are economically practicable.

# GRAVEL-PACKED WELLS*

By Herbert L. White

(*Sanitary Engineer, Physical Plant Department, University of Illinois, Urbana, Ill.*)

We often hear the present age referred to as the "Progressive" period, and we usually think of the methods of construction as highly developed. However, the art of constructing gravel-packed wells still can be improved materially, and we, as superintendents of water works and as engineers, should do all we can to improve the general knowledge on this subject. The superintendent of a water works is able to obtain first-hand information in regard to the weak points in the construction of gravel-packed wells since he is responsible for their maintenance; and the consulting engineer, because of the nature of his business, is able to collect and to distribute information from a wide territory, representing all kinds of drilling conditions and methods of construction.

I should like to relate some of the difficulties of which I have learned concerning wells drilled into sand and gravel, and to raise some questions concerning various design features. In the discussion you will note that I have omitted the names of drillers, because I feel that the results obtained are typical of the times mentioned and are not the fault of the particular drillers.

In the early years, the well driller frequently accepted all the responsibility and constructed wells on the basis of a guaranteed water supply. In such transactions, the engineer either was excluded entirely or else had his hands tied by the very nature of the agreement drawn by the well driller.

## EARLIER UNSATISFACTORY WELLS

Many unsatisfactory wells were drilled in these years. For instance, one competent engineer gave a list of faults which appeared in a well drilled in 1917:—(1) the well was too close to a building;

* Presented before the Illinois Section, April, 1936.

(2) the well caved at the top and was the cause of the building settling; (3) no outer casing was placed in the well to exclude surface water; (4) the gravel packing was not placed uniformly; (5) the gravel was too large in size; (6) sand was discharged with the water; (7) the pumping rate during the acceptance test and during regular operation was too high; (8) the plug in the bottom was not secured properly and came up; (9) the casing was too thin; (10) the top of the well was in a pit and was not sealed to prevent water on the floor or water which might back up from the sewer from flowing back into the well.

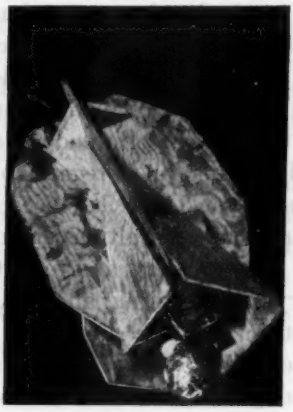

FIG. 1

FIG. 2

FIG. 1. Kodak and photo-flash assembly used in taking pictures inside well casings

FIG. 2. Break in well casing

A history of another well drilled about eight years later as a guaranteed well water supply is of interest because some improvement was made in the method of construction. Much room for improvement remained, however, for although it delivered about 600 gallons per minute, it also delivered some sand for a few months when new, and several years later, for a period of a few months, delivered mud each time the pump was started. The pump was removed after seven years of service and at that time it was observed that a "dog leg" existed above the water surface in the well. The deflection from a straight line was approximately equal to one diameter and was sufficient to compel the mechanics, when replacing the pump, to use a board like a shoe horn to force the flanges on the

drop pipe into the well. Other unexpected things were observed as time passed, such as the presence of a vacuum between the inner and outer casings, a chemical analysis which revealed that shallow water was entering the well, and finally gravel which appeared in the pump discharge. It is unnecessary to say that the pump refused to run about this time. The pump was removed with considerable difficulty because gravel which entered the well above the pump bowls had formed a gravel lock between the casing and the bowls. Whenever the hoist was stopped, the tension in the block and tackle and the compression in the wooden tower caused the pump to continue to rise. After the pump was removed from the well, a kodak was run in the well (fig. 1) and pictures were taken at intervals from the dog leg to the water surface. The pictures revealed that the inner casing had parted at a joint by shearing rivets in a butt strap (fig. 2).

The well was explored by a well driller and several things were learned:

(1) The bottom of the well was about 20 feet higher than was recorded; (2) some of the screens, if the original log was correct, were opposite a clay or silt, rather than opposite a sand formation; (3) a small leak was present in the outer casing; (4) a cave of substantial extent existed at the bottom of the outer casing; (5) another break in the inner casing was present somewhere below the water level; (6) the main screen section was out of water when the pump was in operation.

## RECENT WORK

As a result of a report upon the University of Illinois water system by Alvord, Burdick and Howson, a new well was authorized in 1934. Prior to the construction of this well, a test hole was drilled. Samples of drillings were saved, and unusual care was used in attempting to collect samples that were representative of the drillings. In spite of this care, sieve analyses of the samples from the drill hole did not agree with those from the permanent well. The samples from the test hole may have contained fine material which fell into the well from above the drill, or it is possible that the bailer removed only the finer material.

The permanent well was drilled by the mud scow method. Although I have seen many wells drilled by cable tools, I had not had the privilege before of observing closely the mud scow drilling methods.

First, a large pit is dug by hand and in this pit are placed two girders which are used as anchors for hydraulic jacks. A conductor pipe, somewhat larger than the outer casing to be used, is placed between the girders, the hydraulic jacks are attached, braced, oil pipe connections made, and then the pit is filled up even with the ground surface. A drilling machine is moved up to the edge of the pit and set up ready for drilling. The pit is covered over with a floor that is used for a drilling platform. Next, a hole is dug by hand inside the conductor pipe and as deep as ground water conditions will permit. The first section of casing and some water are placed in the conductor pipe, and drilling is started. The scow is a very heavy bailer, and is moved up and down in the pipe just as cable tools would be. The great weight of the scow and the hydraulic action of the flap valve are very effective during the drilling operation. When the scow drills a short distance below the casing shoe, the driller forces the casing down by the use of the hydraulic jacks. The jacks are operated by oil discharged from a special pump driven by the engine on the drilling machine.

Drilling is stopped when the top of the casing is a foot or two above the derrick floor, and a new length of casing from 7 to 10 feet in length is added. The new section is fastened to the first section by riveting and welding to a butt strap attached to the top of the old section. After the outer casing has been placed to the desired depth, an inner casing with a screen of the proper length is placed inside. The scow is worked in the screen. Packing gravel is added to the annular space between the inner casing and the outer casing. As the screen sinks downward, due to the action of the scow, the packing gravel is supposed to follow and form a gravel envelope around the screen to filter out fine sand.

Two important details in the construction of a well of this type are the variation of the center line of the casing from a vertical line, and the variation of the center line from a straight line. In other words, the well may be inclined, may be crooked, or may be both. Unless one has witnessed tests where wells were plumbed frequently as the construction progressed, or has followed the subject closely, it is surprising to learn how much wells vary from a straight and vertical line. To illustrate this point, I might tell a story about a driller who had had many years experience. He was told that another driller had drilled a well a certain number of inches out of vertical. He replied that the man who could not drill a vertical

hole was no driller, and asked to show what he could do. The driller was given a contract to drill a well, and when he finished, the well was about 30 inches out of vertical. He then changed his story and said that any one who said they could drill a vertical hole was not a driller but a liar.

A crooked hole is very objectionable if it is sufficiently out of line to cause a bend in the discharge column or drive shaft. However, if the well is straight but is inclined only a little, the pump base can be tilted slightly to make it perpendicular to the axis of the well. In fact, some might argue that there will be less damage to the screen or casing in a well if the pump rested against it than if the pump hung free but close enough to it so that the vibration set up when the pump was in operation could cause it to pound the casing or screen.

In a well drilled recently, drillers tried to control the direction of the casing by shifting the mast on the drilling machine with the hope that the scow would strike more often on one side of the hole and cause the casing to go in that direction. They also tried supporting one side of the casing and letting the other side hang free while drilling. This procedure seemed to have little effect on the direction of the hole. The most effective attempt to change the direction of the casing consisted of wedging the top over to one side. In fact, after watching drillers make two gravel packed wells, I wondered just how much we were justified in expecting in the way of a vertical well. During construction the sand and gravel probably are partially suspended by the agitation of the scow, just as filter beds are suspended while back washing. A screen under such conditions might sink straight down, but on the other hand, the shoe on the screen might strike the side of a boulder and be forced quite far out of line. If the screen were supported at the top, at the same time that the surrounding formation was agitated, it is possible that it might hang straight and vertical, but on the other hand, a boulder along one side of the screen would have a tendency to force it out of line. The process seems to me to be largely a matter of chance. With more expensive methods of construction and with control by instruments, there is no doubt that the well could be made almost vertical just as caissons are made. However, in obtaining a well by such methods, the cost will increase, and I wonder if the advantages of such a well is sufficient to offset the increased cost.

Another point of interest is the amount of sand which will be permitted in water pumped from the well. Sand, if present in too large a quantity, will cut the pump bowls, line bearings, valves, etc., and may fill the pipe lines. In fact, sand sometimes cuts entirely through the outer walls of the pump. Most specifications state that the water shall be practically free from sand. When the University had a well under construction, my superior asked me how much sand I thought should be permitted in the water. When I told him I didn't have the slightest idea, because as far as I knew there were no figures in literature to indicate what amount of sand could be permitted, he said that since I would have to decide some time, I might as well decide in advance, and thus prevent a misunderstanding as to what we expected. In two wells which were drilled recently, the sand obtained varied from 0.7 p.p.m. to 9 p.p.m., as measured at the end of the first eight hours of a pumping test.

## PROPER SIZE OF GRAVEL

The size of the gravel used for the envelope around the screen, the amount of gravel, and the size of the screen opening are other items of importance. Should not the size of packing gravel depend on the size of the sand found in the formation? In other words, should not the gravel be selected so that a certain percentage of the native sand will be drawn through the gravel envelope and that another certain percentage of the native sand will be retained? After the size of the packing gravel is fixed, should not the size of the openings in the screen be just as large as possible and still not permit the gravel to pass through?

A combination proposed for use in one well is as follows:

| | |
|---|---|
| Effective size of sand | 0.26 mm. |
| Effective size of gravel | 6.0 mm. |
| Opening in screen | 2.0 mm. |

This seems to be a peculiar combination, since the size of the gravel is twenty times the effective size of the sand, and three times the size of the screen openings. Since it is assumed that the packing gravel forms an envelope around the screen, would it not be wise to specify that a definite amount of gravel be added as a minimum which would be sufficient to form an envelope equivalent in cross section to the annular space between the inner and outer casings? There is a possibility that the well will quit taking gravel and stop yielding sand when surging, before the minimum quantity to form

an envelope is introduced. If such is the case, I wonder just what good is accomplished by the gravel envelope. More gravel could be added if the screen were to be supported from the top, when the screen shoe was still a few feet above the bottom of the sand and gravel formation. The well could be agitated with the screen in this position until the packing gravel could be bailed out at the bottom along with the native sand and gravel.

### CASING MATERIALS

The material used as casing for wells of this character is often a subject for debate. Wrought iron has been accepted as the most desirable material in the past, but from articles which appear in current literature and from some observations I have made, I wonder if it is worth the additional cost over steel. The casings are often too thin, and the rivets at the transverse joints too few. These small items are often overlooked, yet I have seen several failures due to these causes. It might be well to specify a type of plug to be used for closing the bottom of the screen. If the drilling contractor is accustomed to using a different type plug, he could submit it as an alternate when he submits his bid.

### FIELD DRAWDOWN

We discuss the circle of influence of wells in the class room, but not many of us have the opportunity to learn about them at first hand. For this reason I was particularly interested in some data which were obtained at the University of Illinois. Water levels were measured in a number of wells that were not in operation, and the resulting levels were referenced to U.S.G.S. datum (fig. 3). These data revealed that the water table had a slope of 3 feet per 1000 toward wells operated by the local water company although they were 4800 feet from the nearest well.

### PUMP CAPACITY

The capacity of the well pumps should be considered seriously. Only too often, we find that we have pumped wells at too high a rate, i.e., caused too much drawdown, and consequently we have suffered heavy expense as the result. It sounds bigger and more striking to talk about pumping 1,000 g.p.m. from one well than to talk about pumping 500 g.p.m. from two wells. Probably it seems foolish to drill a second well when one can supply the entire demand

when pumped at a high rate, but the latter policy is usually unwise, for several reasons:

(1) It is difficult to maintain a constant supply of water under such conditions, because inevitably repairs must be made to the well pump, the well, or to the equipment affecting the well. For instance, a small gravel-packed well drilled recently was equipped with a water-lubricated deep well turbine pump, which had a capacity much too large for the well. Before many months, the pump pulled the water surface in the well down to the point where the pump broke suction. Then, since water was not present to lubricate

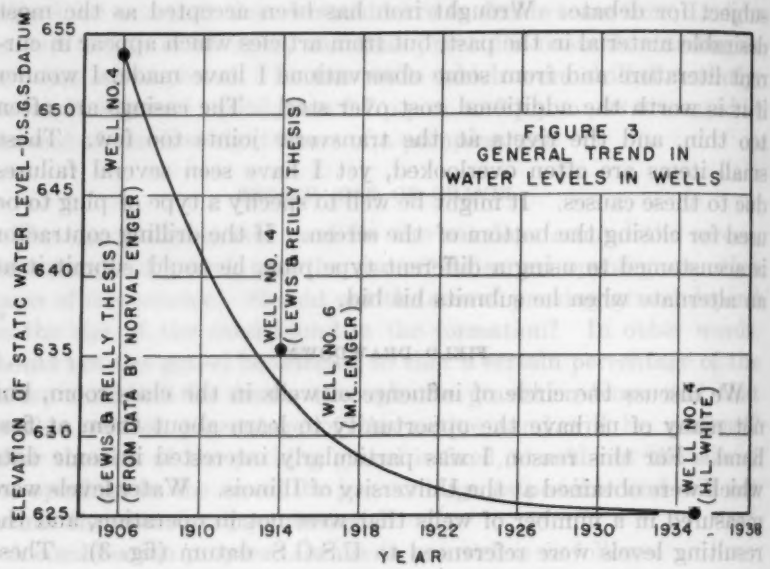


FIG. 3

the pump, the line shaft froze in the bearings and twisted in two. Consequently, the owner was without water from his well for many days.

(2) There is a strong possibility that the screen and the gravel adjacent to it will become clogged by deposits of calcium carbonate, and greatly shorten the life of the well.

(3) High pumping rates cause higher velocities through the gravel pack and that results in more sand being carried into the voids in the gravel which in turn speeds up the velocity through it, etc., until finally the well has to be placed on the retired list.

## NECESSITY OF INSPECTION

One reason that the art of drilling gravel-packed wells has not advanced more is that we have not seen to it that a competent inspector has been placed on each well construction or repair job, not only to see that the terms of the contract are fulfilled, but also to learn more about what can and cannot be done, and to learn what is happening below the ground. For instance, we ought to know if the well mentioned previously with the "dog leg" was left in that condition by the drillers or if the "dog leg" was caused by a cave later, and if the latter, what defect in construction caused it.

There is no doubt but that the engineer is placed in an uncomfortable position in preparing specifications for gravel-packed wells, for he must decide if:

(1) he wishes to design the well completely and accept all responsibility for the outcome;

(2) he wishes to place all the responsibility for the success of the well upon the well driller;

(3) he wishes to adopt some combination of the above two conditions.

We have not accepted the responsibility for design in many instances simply because we have not learned what can reasonably be expected. Better coöperation will be obtained from the well drillers and valuable assistance will be given our inspectors if we make up our minds in advance as to what we expect, make certain that these requirements are reasonable, and then state them clearly in the specification.

Usually discussions of this sort are terminated by a conclusion which gives the answers to the perplexing questions; however, in this case, all I can do is to express a hope that a study will be made in the laboratory of the different variables, in order that the consulting engineer may have more information at hand to use in designing a well. The means used for collecting samples in test holes should be studied more, in an effort to obtain more representative samples. It is possible that, by using shorter lengths of casing, the casing can follow the drill more closely. If we expect certain unusal care in this respect, we should indicate what is required in definite terms. Finally, we can all help ourselves and each other by furnishing more and better inspection for well construction and well repair work to collect every bit of data available.

## *DISCUSSION*

WINFRED D. GERBER (*Engineer, Illinois State Water Survey*): Mr. White's paper is instructive in that he points out certain undesirable results frequently obtained during the construction of gravel packed or gravel walled wells and then admonishes the well contracting and engineering fraternity to do some hard thinking to the end that better wells may be constructed. He calls this the Progressive Age and rightly so. For many years the output of a well was limited to the pump capacities then available. Deep well cylinders had a very definite limitation and while centrifugal pumps were available they were used as *pushers* and not *suckers*.

The multiple-stage deep well turbine pump appears to have been the great incentive to big well construction. Prior to its advent a well that would receive an 8 inch or 10 inch deep well cylinder was a big well. It is not unusual to find records of wells drilled in the early day that were cased at the top with 8 inch and sometimes 6 inch casing, that were a quarter mile or more deep with bottom bores of 3 inches or so. Today it is not uncommon to find wells cased at the top with 26 inch O.D. (about 24 inches inside diameter) casing and finished at depths of 2,000 to 2,800 feet with bore diameters of of 12 inches to 15 inches. The drilling of such wells involve the handling of tools of great weight, therefore, the rigs must be much superior to those of former days. Incidentally, greater investments by the contractor are necessary. This paper by Mr. White on "Gravel Packed Wells" does not permit of discussion on deep well drilling, hence, close adherence to the title is necessary. Gravel packed or gravel walled wells may be divided into two general classes—*small* and *large*. These terms are not very definite in their designation for there is a twilight zone between them.

### SMALL WELLS

Small wells may, however, be designated as those which are 6 inches and 8 inches in diameter and require comparatively light rigs and equipment.

Such wells may be subdivided into: (1) wells where the bore and casing are extended entirely through the aquifer; (screen is then inserted at the bottom and casing pulled back to expose the screen. A certain amount of development is usually desirable to wash in and remove fine sand in the formation close to the screen). (2) Drilling to aquifer, installation of screen and forcing it into the aquifer by

the so-called "bailing in process." In the first case the screen used very likely will have a closed bottom while in the second case the screen will simply be a tube through which the bailer can operate.

LARGE WELLS

Large wells may be designated roughly as those having casings and screens of 10-inch diameter or better.

Such wells may be divided into three general classifications according to method of construction.

1. Well drilled with large diameter, say 24 inches, through the aquifer, the casing extending to the bottom. A smaller casing with screen on the lower end, say 12 inches in diameter, is then set and the annular space filled with gravel. Some development is usually attempted which may or may not be desirable, depending on the character of the aquifer and the size of the gravel pack.

2. Well drilled with large bore, say 24 inches, to the top of the aquifer. Smaller casing, say 12 inches, with screen on bottom is placed in center and extended through the aquifer. Gravel is placed in the annular space between the two and by development, the sand is pulled through the screen and the gravel feeds down around it to form a pack.

3. The mud scow form of construction described by Mr. White is a similar method except that the outer casing was 36 inches and the inner casing and screen was 24 inches in diameter. As was pointed out in the principal paper, the casings went out of line. To insure a straight and plumb job the outside casing must be set so as it acts as a guide for this inner casing. These pipes are not very stiff and when once started on an erratic course it is next to impossible to straighten them up except by pulling back and getting a fresh start. In work of this character, constant supervision should be maintained to check lateral tendencies of the casings and apply correction methods before a definite deflection has been started.

4. A modification of 3 and 4 is the method of setting the inside casing in a well constructed with rotary tools, the bore of which is maintained by what is termed "Mud Walling." A syrupy liquid of mud and water is pumped through a longitudinal hole in the tools and flows up the outside bringing up in suspension the cuttings. When the well has been completed, the casing and screen are inserted. By development, the mud is washed in along with fine sand while the gravel pack flows downward to envelop the screen.

5. Caisson method. There are several modifications of this type of construction, but in its simplest form it consists in excavating a well bore of relatively large diameter, 40 inches or better, entirely through the aquifer, the bore being maintained by a sheet steel pipe or curb. When the full depth has been reached, the bottom is sealed with a concrete plug. The screen and inner casing are then assembled and installed and the annular space filled with selected gravel. The outer casing is removed as the gravel filling progresses. This method is the most expensive but it permits a flexibility in the grading and placing of the gravel. A modification of the caisson method is the construction of screen and well curb on either a steel reenforced concrete or wood shoe. As the interior material is removed the whole structure settles. This method is better adapted to large diameter, shallow wells than for smaller sizes at depths of 100 feet or better.

The whole idea in these various forms of construction is the production of an infiltration zone much larger in diameter than that of the metal screen which restrains it. It is hoped that a sufficient volume of gravel will be distributed evenly about the metal screen so that the velocity of the entering water will be so low that sand will not be drawn into the artificially placed gravel. This situation can always be accomplished if the well owner does not become too ambitious for a large yield. In wells constructed in fine sand aquifers, it is desirable to keep the entrance velocities as low as possible. This generally means large diameter wells, within reason, and low capacities. Hydraulically, it is preferable to pump two or more wells at low rates than one well at a high rate.

## GRAVEL SPECIFICATIONS

It is generally conceded that the gravel used for packing should be selected. Selected for what? Well, it appears that it should be selected for size, shape, character and condition.

1. Size. The size or diameter should be a function of the size of the sand that is to be screened out. If three marbles of equal size are placed in contact, the pore space will admit the passage of a fourth sphere having a diameter of about $\frac{2}{15}$ of the larger ones. Therefore, the gravel used in the pack should be of such a size that the pore space will be too small to permit the sand grains from entering and thus restricting the flow channel. As an illustration, if the natural sand grains had a diameter of .04 inch, the pack

gravel could have a diameter of .25 inch, or if the sand was very fine, say .005 inch, two sizes of pack gravel might be used, .03 inch and .20 inch approximately. The screen slot opening should be of a width that the pack gravel will be excluded.

2. Shape. The ideal shape of pack gravel is a sphere. Flat and irregular shaped gravel or crushed stone should not be used. Glass marbles or beads would be quite ideal for shape. Carefully graded "Bird's-eye gravel" or roofers gravel should prove quite satisfactory.

3. Character. Gravel for well packing should be of hard granite like material. Shale or limestone gravels are not suitable.

4. Condition. The gravel should be clean washed and of uniform size. Variation in size has the effect of reducing the pore space.

## SAMPLES

The original paper mentions sample collection. This phase of well construction is something that can be given much attention by both the contractor and the engineer, for it is upon a correct set that the entire planning of the well is based. Bailer samples frequently do not represent true conditions. A device for collecting samples should be a simple operating affair designed and used for that purpose only. The mental attitude of the driller has much to do with the correctness of samples. The method of measuring the depths frequently is open to criticism, all of which means that the development of a satisfactory technique is highly desirable and will require the best efforts of all concerned.

## DIFFICULTIES

There are two principal difficulties in the development of satisfactory well construction. The first has to do with the engineer and the second has to do with the contractor. In a majority of cases the consulting engineer has but a few well jobs during his entire experience. There are very few who specialize in well design. The result is that lack of experience reacts against him and is displayed in the specifications. The contractor on the other hand has for so many years conducted his business without engineering supervision that he resents any intrusion and publicly belittles the efforts of the engineer. Very likely the contractor is not to be blamed for this attitude.

However, this is an era of progress and development in drift well construction is on the order sheet. It, therefore, behoves the engi-

neer to study carefully the art of well construction and the contractor in turn should view the situation with an open mind so that between the two a satisfactory piece of work will result. The public cannot expect the contractor to take all the risks without paying the price neither should the contractor assume that improvement in technique cannot be indulged in and still produce a profit. Many of the difficulties which frequently result in the loss of reputation can be traced directly to a lack of fundamental knowledge. It is inconceivable to hope that knowledge may some day be available covering every foot of the ground surface and to depths of one-half mile or more but by systematic and persistent efforts and through close cooperation much can be achieved. The greater the volume of basic data, the less will be the risk and the more will the customer sing the praises of all concerned.

# ARTESIAN WATER IN MINNESOTA AS ILLUSTRATED BY THE TWIN CITY ARTESIAN BASIN*

By G. M. Schwartz

*(Department of Geology and Mineralogy, University of Minnesota)*

A large state such as Minnesota may show great variation in the type and structure of rock formations which are found beneath the surface and a brief examination of the large scale geologic map of the state, published by the Minnesota Geological Survey[1] shows that Minnesota does indeed have a complex geology.

## GEOLOGY OF MINNESOTA

In a general way the state may be divided into three geologic zones. The first is an area where igneous and metamorphic rocks are exposed or occur beneath the unconsolidated mantle of glacial drift. This includes all of the northeastern part of the state and a considerable area in the central part and along the Minnesota river valley. Igneous rocks have solidified from a molten condition and metamorphic rocks have been formed from pre-existing igneous and sedimentary rocks by heat, pressure, and movement. It is to be expected that both of these classes of rocks would have a low porosity and, therefore, yield little water. Artesian water is, therefore, rare in these rocks of low porosity, although sometimes found in fracture zones or in the porous upper portions of lava flows. From the point of view of extensive artesian supplies we may eliminate this area from consideration.

The second geologic zone is a series of areas in western Minnesota,[2] where relatively thin deposits of Cretaceous sandstones and shales are found beneath the glacial drift and lying on the igneous and metamorphic rocks below. At places, especially near Marshall, the sand-

* Presented at the Minnesota section meeting, November, 1936.

[1] Available at the University of Minnesota Press, Minneapolis, Minnesota. (Price $1.00).

[2] The occurrence of water in the northern part of this area is described in Minnesota Geological Survey Bulletin 22.

stones are continuous enough to give a local artesian basin. In this area the glacial drift is very frequently thick and local supplies of artesian water may be obtained by tapping the more continuous sand and gravel beds. Throughout this region wells pass into the igneous and metamorphic rocks at depths of a few hundred feet and the same conditions encountered in the first zone prevail.

*Rock formations of the Minneapolis-St. Paul Area*

| PERIOD | FORMATION | AVERAGE THICKNESS (IN FEET) | APPARENT RANGE IN WELL LOGS (IN FEET) |
|---|---|---|---|
| Recent | River alluvium | | 0–150 |
| Pleistocene | Glacial drift, etc. | 100± | 0–400 |
| Ordovician | Galena | Top eroded | 0– 20 |
| | Decorah shale | 25 | 0– 25 |
| | Platteville limestone | 30 | 25– 35 |
| | Glenwood beds | 5 | 2– 7 |
| | St. Peter sandstone | 158 | 145–165 |
| | Shakopee dolomite | 45 | 35– 60 |
| | New Richmond sandstone | 11 | 5– 15 |
| | Oneota dolomite | 80 | 70– 90 |
| Cambrian | Jordan sandstone | 90 | 80–105 |
| | St. Lawrence formation | 180 | 160–200 |
| | Franconia sandstone | 65 | 45– 80 |
| | Dresbach sandstone | 155 | 100–200 |
| Cambrian ? | Hinckley sandstone | 220 | |
| | Red Clastic series | 1012 | (Lakewood well) |
| Pre-Cambrian | Basalt flows, granites, etc. | Unknown | |

The third zone consists of the southeastern part of the state including a narrow belt extending as far northward as Duluth. This area is underlain by a series of sedimentary rocks mainly of Paleozoic age and forms the main artesian zone of the state. The series of limestones, sandstones and shales in this area were originally deposited as a series of marls, sands and muds in arms of the ocean which existed over the continent in Paleozoic time. To the north and northwest the igneous and metamorphic rocks probably formed

a highland and the beds now dip gently away from this highland and the water which flows into the rocks in their outer margins moves down the structure into the basin and fills the porous beds (principally sandstone), and becomes the source of artesian water which may be obtained in large amounts throughout most of this part of the state. A general discussion of the occurrence of water in the southern half of the state is given in United States Geological Survey Water Supply Paper 256, published in 1911. This area is now being restudied by Professor G. A. Thiel of the Minnesota Geological Survey, to bring the data up to date.

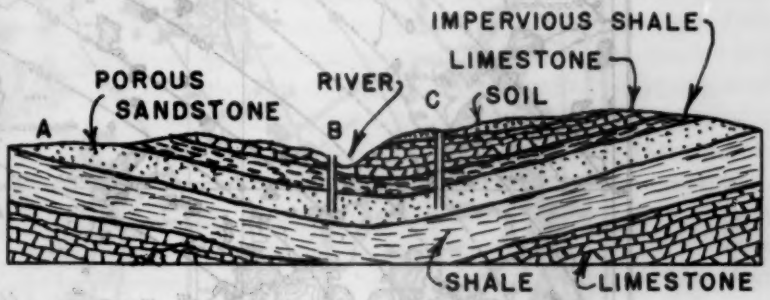


FIG. 1. DIAGRAM OF AN ARTESIAN BASIN, SHOWING THE SOURCE OF WATER AND THE CONDITIONS NECESSARY FOR FLOWING AND NON-FLOWING WELLS

## THE TWIN CITY ARTESIAN BASIN

The occurrence of artesian water in this area is well illustrated by the Twin City Artesian Basin which has been worked out in some detail because of the importance of the metropolitan area.[3]

The rocks found in the metropolitan area are shown in the table on the opposite page, with the average thickness based on a considerable number of wells. To the southward these thicknesses vary

[3] The geology, including the occurrence of water in this area, is completely described in Minnesota Geological Survey Bulletin 27, including 52 maps and other illustrations. Obtainable at the University of Minnesota Press (Price $3.50).

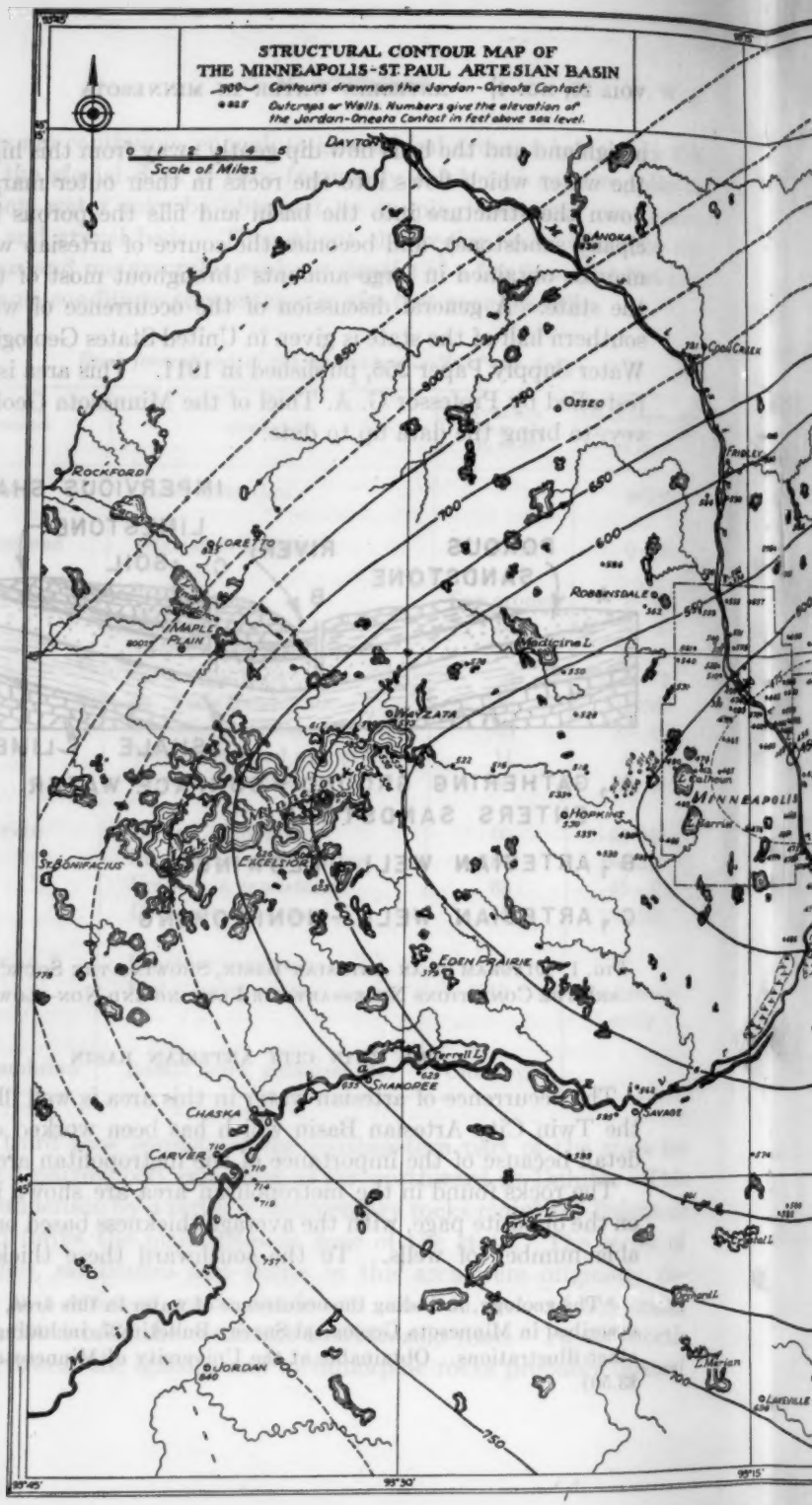
STRUCTURAL CONTOUR MAP OF
THE MINNEAPOLIS-ST PAUL ARTESIAN BASIN
Contours drawn on the Jordan-Oneota Contact.
Outcrops or Wells. Numbers give the elevation of the Jordan-Oneota Contact in feet above sea level.
Scale of Miles
0 1 2 3 4 5
DAYTON
ANOKA
OSSEO
ROCKFORD
ST LORETTO
MAPLE PLAIN
ROBBINSDALE
MINNEAPOLIS
HOPKINS
ST BONIFACIUS
EXCELSIOR
EDEN PRAIRIE
SHAKOPEE
SAVAGE
CHASKA
CARVER
JORDAN
LAKEVILLE
700
750
800
850
900
93°45'
93°30'
93°15'

FIG. 2

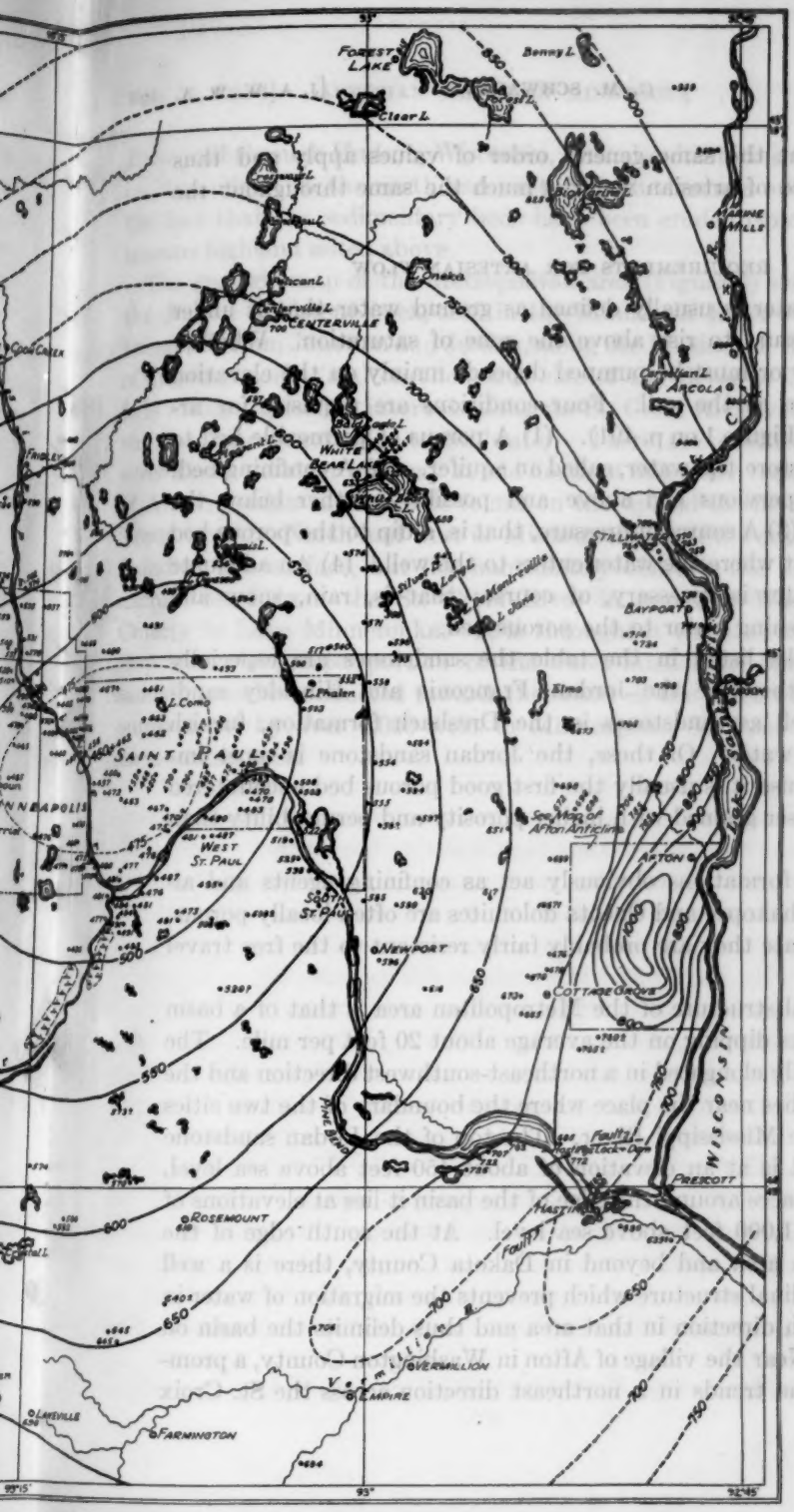
FOREST LAKE
Clear L.
Bonny L.
Marine Mills
Oneka
Duncan L.
Centerville L.
CENTERVILLE
Goose Creek
Turtle L.
Gem L.
WHITE BEAR LAKE
Silver L.
Long L.
L. Josephine
L. Jane
Carnelian L.
ARCOLA
Stillwater
Bayport
Elmo
L. Phalen
L. Como
SAINT PAUL
MINNEAPOLIS
WEST ST. PAUL
SOUTH ST. PAUL
NEWPORT
Axis of Afton Anticline
AFTON
COTTAGE GROVE
Fault
Fault
PRESCOTT
WISCONSIN
PINE BEND
ROSEMOUNT
VERMILLION
EMPIRE
LAKEVILLE
FARMINGTON
550
600
650
700
750
800
850
900
1000
93°15'
93°
92°45'

FIG. 2

somewhat, but the same general order of values apply and thus the occurrence of artesian water is much the same throughout the zone.

### REQUIREMENTS FOR ARTESIAN FLOW

Artesian water is usually defined as ground water that is under sufficient pressure to rise above the zone of saturation. Whether the wells flow or must be pumped depends mainly on the elevation of the surface at the well. Four conditions are requisite for artesian water (Figure 1 on p. 491). (1) A porous or permeable bed to conduct and store the water, called an aquifer. (2) A confining bed, that is an impervious bed above and possibly another below the porous bed. (3) A source of pressure, that is, a dip to the porous bed from the point where the water enters to the well. (4) An adequate supply of water is necessary, of course; that is, rain, snow and streams furnishing water to the porous bed.

In the rocks listed in the table the sandstones are especially porous, and therefore the Jordan, Franconia and Hinckley sandstones, as well as sandstones in the Dresbach formation, furnish most of the water. Of these, the Jordan sandstone is most important, because it is usually the first good porous bed encountered and it is coarser grained with higher porosity and permeability than the others.

The other formations obviously act as confining agents and although the Shakopee and Oneota dolomites are often locally porous, in the aggregate they are probably fairly resistant to the free travel of water.

The general structure of the Metropolitan area is that of a basin with the rocks dipping on the average about 20 feet per mile. The basin is slightly elongated in a northeast-southwest direction and the lowest point lies near the place where the boundary of the two cities intersects the Mississippi River. The top of the Jordan sandstone at that point is at an elevation of about 450 feet above sea level, whereas at places around the edge of the basin it lies at elevations of from 800 to 1,000 feet above sea level. At the south edge of the Metropolitan area and beyond in Dakota County, there is a well defined anticlinal structure which prevents the migration of water in a north-south direction in that area and thus delimits the basin on that side. Near the village of Afton in Washington County, a prominent anticline trends in a northeast direction across the St. Croix

River and through Hudson, Wisconsin, thus also delimiting the basin on that side. On the north and west sides the basin is outlined by the fact that the sedimentary beds have been eroded from the old igneous highland noted above.

The geologic map of the Metropolitan area (Figure 2) shows that the Jordan sandstone is exposed in the Minnesota River valley between Merriam Junction and Jordan, along the Mississippi River east of Hastings locks, and on the south side of the River at Nininger. It is widely exposed at intervals along the St. Croix River valley from the Soo Line drawbridge to Stillwater. There are excellent exposures along the river highway at and north of Stillwater. Southwest of Afton, Jordan sandstone is found on the upland in the valley of Trout Creek. North and west of Minneapolis and St. Paul it is not found at the surface, but lies immediately beneath the drift in a narrow band from the St. Croix River in northern Washington County to Lake Minnetonka. It is throughout these areas of outcrop that the greater portion of the water must enter the Jordan sandstone to be conducted toward the center of the basin where it is pumped out by over 1,000 wells in Minneapolis, St. Paul and the surrounding area.

# THE EFFECT OF MODERN SEWAGE TREATMENT ON THE SOURCES OF PUBLIC WATER SUPPLIES*

By K. L. Mick

*(Chief Chemist, Minneapolis-Saint Paul Sanitary District)*

## INTRODUCTION

One of the features of American water works practice is the extensive use made of surface bodies of water as sources of public water supplies. It has been estimated that surface waters supply at least two-thirds of the total volume and serve two-thirds of this country's population using public water supplies. However, the number of public water supplies from surface sources is less than the number originating in wells.

The preservation of our surface waters for continued use as public water supplies becomes a problem of ever increasing importance. The population of the world doubled between the years 1800 and 1900, and the current rate of growth indicates that the world population will at least have doubled again during the century ending with the year 1999. The relation of these figures to water supply, sewage disposal and general sanitation must profoundly affect the future of the human race. Water purification in its modern development of efficiency in this country is only about thirty years old. The knowledge of methods of water purification and sewage treatment have now advanced to a relatively high efficiency but the application and adoption of these methods throughout the world has, unfortunately, been very slow, especially in respect to sewage disposal.

## EXTENT OF SEWAGE TREATMENT IN THIS COUNTRY

Prior to 1890, little attention was given to objectionable conditions produced by the discharge of sewage into American rivers and lakes. Even as late as 1935 it was estimated that the total sewage and industrial waste pollution load in this country was being reduced by treatment by only about 20 percent. In 1935 it was estimated that

* Presented before the Minnesota section, November, 1936.

only a little more than half of the total population of the United States was served by a sewer system. The National Resources Committee Report on Water Pollution, dated July 1935, states that there are ninety-three cities in the United States with more than 100,000 population. The total population of these ninety-three cities is approximately 36,000,000. The sewage from 68 percent of this population is untreated, while 17 percent receives primary treatment only and 15 percent receives further treatment. However, the stimulating effect of the P.W.A. on sewage disposal at the present time will no doubt result in considerable improvement in the figure given above in the next few years.

At the present time it may be said that the volume of sewage in the United States receiving treatment is increasing much more rapidly than is the population contributing sewage. It may be of interest to mention a few specific examples of sewage and water treatment problems in this country.

The cities of Buffalo and Detroit are now preparing to construct sewage treatment plants. Bacteriological efficiency will be paramount to protect downstream water supplies. Very great dilution in the Niagara and Detroit Rivers obviates any necessity for complete treatment. At present Niagara Falls takes water from the Niagara river a few hours after the water receives the sewage from more than 500,000 people at Buffalo. The flow of the Niagara River is about 200,000 cubic feet per second. Despite this high dilution the danger from the raw water has been very great and Niagara Falls typhoid death rate for the ten year period from 1899 to 1908, prior to the installation of a water purification plant, was 133 per 100,000, the highest in the country. After the installation of the water purification plant the typhoid death rate dropped to 5 per 100,000. Although the water purification plant had apparently minimized the danger, the following occurrence will serve to illustrate the necessity for sewage treatment in addition to water purification in this case. In March, 1933, it became difficult to maintain residual chlorine in the water supply because of sudden rises in the chlorine demand. These flashes of pollution were traced to sudden releases of sewage laden waters which had been locked in the Buffalo harbor by ice. The pollution passed out into Lake Ontario and traveled unbelievable distances to affect even the water supplies at Rochester and Oswego.

The cities of Louisville, Cincinnati and Pittsburgh on the Ohio

River have no sewage treatment. All possible practicable measures have been taken along the river to extend water purification to its limit of effectiveness, but very little progress has been made with the installation of sewage treatment plants. It becomes a question of how much longer the water plants can handle the load. The Ohio River furnishes water supplies to over thirty cities having an aggregate population of about 2,000,000. In U. S. Public Health Bulletin No. 143, the following opinion is expressed regarding sanitation along the Ohio: "in the control of pollution on the Ohio River the protection of water supplies is paramount, nuisance conditions being negligible because of the high dilution. The objective should evidently be to limit the burden placed upon the water treatment plants, allowing them an ample margin of safety, without unduly increasing their cost of construction and operation. The rate of increase in sewered population is such that, unless some altogether unexpected advance is made in the efficiency of water purification processes, the time must come when the purification plants will be overtaxed by the steadily increasing sewage pollution."

At East Chicago, Hammond and Whiting, the pollution load in Lake Michigan is so great as to be a menace even to well constructed and well operated water filtration plants. A quarter of a mile from the new Hammond filtration plant raw sewage from the north section of Hammond and the city of Whiting discharges into the lake. Although the new water purification plant at Hammond will increase the safety of its water supply immensely, the superintendent of the plant states that: "The water purification problems of Hammond and nearby cities will be greatly simplified some day, however, by the construction of modern sewage treatment plants. More satisfactory disposal of sewage and industrial wastes will greatly reduce the hazard and cost of water purification."

The history of sewage treatment and water purification at Milwaukee is another example of the inter-relationship of the two problems. Milwaukee reached a population of 100,000 before a public water supply was provided. The first intake was constructed in 1873 and extended 2100 feet offshore in eighteen feet of water. In 1895 a new intake was completed extending 1.5 miles from shore in sixty feet of water. The water from the new intake was much superior to that from the old one, but periodically and with increasing frequency, as time passed, Milwaukee suffered epidemics of water-borne diseases. This resulted in the employment in 1911 of the Milwaukee Sewerage

Commission which, although primarily changed with the study of Milwaukee's sewage treatment problem, nevertheless recognized the close relationship between sewage treatment and the protection of the water supply. The Commission recommended that, in addition to the installation of sewage treatment facilities, a water filtration plant should be provided immediately. But nothing was done with respect to water purification. A decision was made to proceed with a sewage plant prior to undertaking water purification and the sewage plant was placed in operation in 1924. Subsequent events have demonstrated that, contrary to earlier predictions, the sewage plant itself did not adequately protect the water supply. This was soon realized and in 1926 the City Engineer began an extensive study of the problem. As a result construction of a 200 M.G.D. water purification plant was started in 1934. Although the sewage treatment plant had improved the quality of the water supply, by-passing of sewage still continued to pollute the lake due to the fact that about 60 percent of the city's area is on a combined sewer system and overflows of sewage are frequent during storms. In addition to the new water treatment works, Milwaukee is also now constructing an addition to the sewage treatment plant. The experience of Milwaukee serves to illustrate the now accepted fact that sewage treatment can be considered only as an aid or protection to the process of water purification, but not as a substitute for it.

## RELATION OF SEWAGE TREATMENT TO WATER PURIFICATION

Although the specific examples have already illustrated some of the factors in the relationships between sewage treatment and water purification, other factors should be mentioned.

Perhaps the most important factor is the relationship between the bacterial efficiency of sewage treatment processes, the natural purification in the water courses, and the permissible bacterial loading on water treatment plants.

It has been commonly stated that the total bacterial count of sewage ranges from one to ten million per cubic centimeter and that the organisms of the coli-aerogenes group, which are indicative of fecal contamination, approximate 100,000 per cubic centimeter. Assuming a daily sewage flow of 120 gallons per capita this would indicate a coli-aerogenes contribution of about 200 billion per capita per day.

Some conception of the large overall bacterial reductions which

must be effected before water from a sewage polluted stream can be sent to the consumer can be obtained by comparing the above figures for the bacterial content of sewage with the most widely accepted standard of bacterial quality of drinking water. This standard is that adopted by the United States Treasury Department and is essentially not more than 0.01 coli per cubic centimeter or 1 coli per 100 cc.

The percentage removals of bacteria in sewage by various methods of sewage treatment have been listed by Whipple as follows:

| | per cent |
|---|---|
| Plain sedimentation | 25 to 75 |
| Chemical precipitation | 45 to 80 |
| Disinfection of settled sewage | 90 to 95 |
| Activated sludge process | 90 to 98 |
| Disinfection of filter effluents | 98 to 99 |

From this it appears that, with a sewage treatment plant in operation, from 25 to 99 percent of the bacteria present in the original sewage can be removed before the plant effluent is discharged into a watercourse, depending on the type of sewage treatment installed.

The bacteria of sewage origin are then further reduced by dilution in the receiving body of water and by natural purification in the water. By way of illustration, if we wish to locate a water treatment plant a certain distance downstream from the source of raw sewage or plant effluent pollution there are several factors which will influence the concentration of bacteria reaching the water supply intake. Some of these factors may be described as follows:

(1) *Pre-treatment.* That is, the bacterial concentration reaching the water supply intake will be affected by the amount and kind of treatment given the sewage before it enters the stream. The reductions to be expected by the various treatment processes have already been given above as ranging from 25 per cent to 99 per cent.

(2) *Dilution.* The bacterial concentration will be reduced by dilution of the raw sewage or treatment plant effluent in the receiving body of water. The amount of dilution available will directly affect the bacterial concentration at the waterworks intake.

(3) *Natural Purification.* The bacterial concentration will be further reduced by natural purification in the water before it reaches the water supply intake. The bacterial concentration at the intake will be dependent on the time of flow of the water from the source of pollution to the intake and on several other variable factors affecting natural purification, some of which will be mentioned here briefly.

### VARIATIONS IN CONCENTRATION OF BACTERIA

It has been frequently observed that the zone of greatest bacterial density in the receiving stream does not occur immediately below the sewer outfalls, but at a point ten to thirty hours flow-time downstream. Moreover, the maximum zone is farther downstream in the winter than in the summer. Thus, conditions of bacterial pollution occurring in streams during the winter season, when natural purification agencies are less active, may impose a more severe burden on a water plant than that prevailing during the summer, depending upon the location of the water plant in respect to the source of pollution.

After the initial increase the number of bacteria tends to decrease at more or less orderly rates depending on the temperature and initial concentration. A 98 percent reduction in bacteria has been often observed in about five days flow-time. Whipple has estimated that *Bacillus typhosus* will die away in water at the rate of about 50 percent in one to three days and 90 percent in three to thirteen days. However, some of the more resistant organisms may live for several weeks. They die more rapidly in heavily polluted and warm water than in clean and cold water.

With the aid of typical die-away curves of coli in polluted waters, it is possible to estimate the dilution and treatment of sewage required to safeguard water supplies. For this purpose it is also necessary to establish some limiting standard of maximum raw water bacterial content consistent with the production of satisfactory final effluents from water purification plants to meet the Treasury Department drinking water standard of one coli per 100 cc. The United States Public Health Service concluded from a rather extensive study in the Great Lakes and Mississippi region that a coli maximum of 5000 per 100 cc. (or 50 per cubic centimeter) would appear to be rational. This marked approximately the critical average density of raw water pollution above which difficulties in effective treatment and in obtaining palatable effluents have been observed quite consistently as beginning to appear. This standard is for water plants having both filtration and chlorination. For raw waters subjected to chlorination only a standard of 50 coli per 100 cc. (5 per cubic centimeter) is recommended.

Some data on the Mississippi River obtained by the Minneapolis-St. Paul Sanitary District may be of interest as illustrating some typical bacterial concentrations. The coli content of the river entering Minneapolis and at the water supply intakes for the Twin

Cities is usually less than 1000 per 100 cc. This is well within the Public Health Service standard of 5000 per 100 cc. for raw waters subject to filtration and chlorination. Contrast this with the average of 12,000 per cubic centimeter observed at the Twin City Lock and Dam about eleven miles below the water intakes for July, August and September, 1934, and 9000 per cubic centimeter for the same period at the Inver Grove Bridge about eight miles below St. Paul. The data for August and September, 1934, show that during the eight days flow time between the Inver Grove Bridge and the Hastings Dam, the total bacterial count dropped from 2,400,000 per cubic centimeter to 30,000 per cubic centimeter, a 98.7 percent reduction in 15 miles of river. At the same time the coli dropped from 11,000 per cubic centimeter to 240 per cubic centimeter, a 98 percent reduction, but still too high for a source of public water supply (24,000 per 100 cc. compared to the standard of 5000 per 100 cc.). In Lake Pepin, about forty miles farther downstream, the coli index drops to about the same as that entering Minneapolis, usually less than 1000 per 100 cc.

No discussion of the effects of sewage treatment on water supplies would be complete without mention of the at-first-thought rather surprising fact that under certain conditions and in some respects the construction of a sewage treatment plant may actually result in impairing the operation of a water treatment plant downstream. In some cases profuse growths of green algae and other vegetation have occurred in streams after receiving biologically oxidized effluents from sewage treatment plants. Such growths die and decompose to create a nuisance, shorten the filter runs in the water plant, and often result in an unpalatable water—all the result of high efficiency sewage treatment.

## COST ASPECTS

From the United States Public Health Service studies, it appears that for each successive increase of 1000 in the average raw water coli index the total cost of purification is increased by approximately one dollar per million gallons, not including fixed charges against construction. The total cost of water purification including construction and operation may vary from $10 to $35 per million gallons, or from about $0.40 to $1.40 per capita per year, according to the degree of pollution of the raw water. The cost of complete sewage treatment usually ranges from about $0.90 to $1.00 per capita per

year. The balance of cost as between sewage treatment and water purification would thus appear to be approximately equal.

## CONCLUSION

In this country there has occurred a steady increase in the pollution of surface watercourses by sewage and industrial wastes. This tendency has been due both to a disproportionate growth in urban population as compared with rural and to the extension of sewerage systems in the existing urban population groups.

Until recently the use of surface streams for the dual purpose of water supply and sewage disposal has been attended by comparatively few serious difficulties. Contributing factors have been the ample length and relatively large volume of our major inland streams and lakes and the rapid advance in the development and application of water purification. During the past thirty years a steady reduction has occurred in water-borne diseases throughout the country, notably in urban communities using purified surface supplies. In spite of this tendency, pollution of the sources of many surface supplies has increased with such rapidity as to suggest the possibility of failure of existing safeguards to provide an adequate degree of protection. The extent to which the availability of these sources may be affected by the progressively increasing pollution is a question of practical importance to all sanitary and water works engineers. The relation of stream pollution to public water supplies must become more clearly defined as time goes on.

The use of sewage treatment as a specific measure for protecting sources of water supply has been justifiably regarded as being secondary to the use of water purification. But with increasing pollution of our river systems and the possible over-burdening of water purification plants, more consideration may be expected to be given to sewage treatment than has heretofore been necessary.

# THE FEEDING OF POWDERED ACTIVATED CARBON*

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The increasing acceptance of activated carbon as one of the major materials used in water purification brings to the fore the necessity of providing the best possible feeding equipment for this purpose in each plant in which it is used. The most suitable equipment for this purpose may, and probably will, vary with the design and size of the plant in which the carbon is to be applied. In order to aid the filter plant operator in deciding upon the proper feeding arrangements, the characteristics of carbon, as related to this problem, are discussed herewith.

## CHARACTERISTICS

Unlike all of the other materials, which are used in treating water and which are regularly fed into the water at various points in the plant, activated carbon is completely insoluble in water. This eliminates at once any idea of utilizing a true solution feed. However, some forms of activated carbon (including the quality which is most widely used in water purification plants), will remain in suspension in water for a long time, so that some so-called solution feed arrangements were adapted for use in the early days of carbon application in water treatment.

In addition to its lack of solubility in water, activated carbon also differs in other ways from other powdered materials which are fed in the water plant. For example, there appears to be a definite relation between surfaces exposed for adsorption and the activity of the carbon. All water carbons are delivered to the plants in a finely divided state, the greater part passing through a 300 mesh screen. This extremely fine division of particles also adds to feed-

*Contributed record of Investigations.

ing problems, just as it may create a dust problem in the vicinity of the machines if not properly handled.

Activated carbon differs enormously from other chemicals fed in the water plant in apparent density, being so light in weight, according to volume, that it arches easily in the hoppers.

Activated carbon is very difficult to wet quickly and has the tendency to float on the surface of the water. Therefore due consideration should be given to the proper design of the solution or mixing pot. A water ejector, whether necessary or not, should be employed following the solution pot to insure thorough wetting of the carbon particles.

## EARLY METHODS

Those who pioneered the feeding of activated carbon in powdered form were anxious to avoid expenditures above what was absolutely necessary; first, because they wanted to prove the value of this material in their plant before making such an expenditure; and second, because the feeding of powdered activated carbon was introduced into water plant practice right at the beginning of the depression period when funds were low and often not available at all for the purchase of equipment. Consequently, early feeding was done in the most primitive way possible. For instance, at Newport News, Virginia, a Coca-Cola barrel was mounted above the sedimentation basins and was equipped with a small mechanical agitator. It was kept filled with a suspension of powdered activated carbon, which was run off to the point of application by gravity through an ordinary spigot. At Lake Forest, Illinois, an ordinary second hand oil drum was cleaned out and used for this purpose, stirring the contents of the drum occasionally to keep the carbon in suspension. (Both of these plants have since installed the very latest types of dry feeding equipment.)

## DRY FEEDING METHODS

Many plants utilized a spare dry feed machine, which they already had on hand, thereby attempting to feed activated carbon in a machine which was originally designed to feed alum or lime or some other chemical, and many other operators, who were unable to secure suitable appropriations for dry feed machines, built ingenious, but not always the most efficient type of feeding equipment of their own. Since the majority of water treatment plants are

using dry feeding equipment, and it appears that dry feeding is obviously so well adapted for that purpose, we will confine our description of feeding equipment almost entirely to that type.

Lately there has been more recognition, by equipment manufacturers, of the problems involved in the dry feeding of powdered activated carbon. It is hoped that more attention will be given so that they will be able to offer machines which are purposely designed for the feeding of activated carbon, and are not just a unit which was originally designed for some other purpose and modified for use in feeding powdered activated carbon.

Especially is it true that the safeguarding of these machines against the hazards created by carbon dust in the air during feeding has been overlooked, and this should be carefully considered by any water plant operator in making installations of dry feeding equipment and machinery for this purpose. Water plant executives, engineers and chemists should carefully consider the problems involved, and should see that all electrical devices located in close proximity to the feeding of powdered activated carbon should be as dust-proof as possible. Such a demand from purchasers of course will automatically be taken care of by the manufacturers of dry feeding equipment, who will be glad to furnish equipment suitably designed when specified. In the past, of course, they have been unable to do so when the requirement has been merely for a low priced machine.

## FUNDAMENTAL RULES

It may be helpful to those contemplating the feeding of activated carbon, or to those who are already feeding carbon who feel that their present carbon feeding equipment is inadequate or might be improved upon, to lay down the following fundamental rules which they should carefully consider in making selection of their dry feeding equipment:

1. Place the problem as completely as possible before the manufacturer of the equipment, giving him possible points of location in relation to raw water inflow, coagulation, mixing and sedimentation basins and filter influent, and all salient features affecting the installation and operation of the equipment, so that he may give you the most intelligent and most complete recommendation and quotation.

2. If possible feed from a floor or room, isolated as far as possible

from the main filter gallery, whereby the dust nuisance will be minimized as fully as possible.

3. If possible make the hopper large enough so that it will not be necessary to fill it too frequently.

4. The hopper should be provided with sufficient agitating or vibrating devices to eliminate any chance of arching or of uneven feeding.

5. Be sure that the motor and mechanical driving equipment is amply powerful to operate to full capacity continuously, without any danger of running hot, and that ample facilities for proper lubrication while operating are provided in the mechanism.

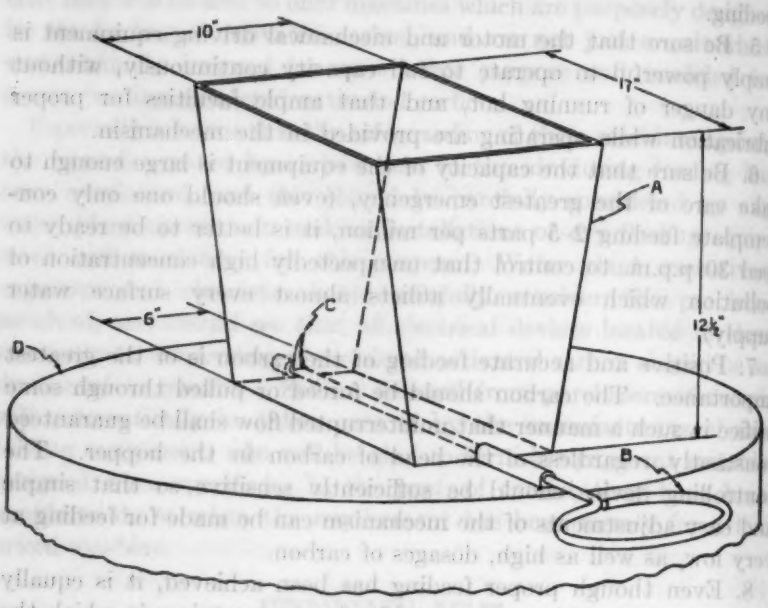
6. Be sure that the capacity of the equipment is large enough to take care of the greatest emergency, (even should one only contemplate feeding 2–5 parts per million, it is better to be ready to feed 30 p.p.m. to control that unexpectedly high concentration of pollution which eventually afflicts almost every surface water supply).

7. Positive and accurate feeding of the carbon is of the greatest importance. The carbon should be forced or pulled through some orifice in such a manner that uninterrupted flow shall be guaranteed constantly, regardless of the head of carbon in the hopper. The controlling device should be sufficiently sensitive so that simple and easy adjustments of the mechanism can be made for feeding at very low, as well as high, dosages of carbon.

8. Even though proper feeding has been achieved, it is equally important to provide a proper orifice box or container in which the carbon may be completely wetted down and carried away immediately. This must be carefully considered in connection with any installation of jets, water sprays, sprinkler heads, or vortex pulls, such as have already been widely used.

9. A good volume of water should be provided for carrying away the carbon to the proper points of application so that there may be no clogging of pipe lines. It is also desirable to see that there are no low points in the distributing line where the water will fail to drain out thoroughly, to avoid settling of carbon in the line or possible freezing of the water in the line during cold weather. Not only is it necessary that an adequate volume of water under ample pressure be available, but also that the discharge lines be of ample size, free from sharp bends and head consuming interferences to free flow.

10. Although an operator may feel that the proper point of application is well established before he starts feeding activated carbon, it is well to provide for feeding at other points in case of emergency, since every water plant is subject to extraordinary conditions at certain periods of extreme and concentrated pollution, which may not be anticipated at the time of installation.




A - Hopper.
B - Cutting Tool.
C - Cutting Blade.
D - Bin-Lid.

FIG. 1

DUSTLESS METHOD FOR EMPTYING BAGS

Since water plants are open to visits at almost any time by consumers and other interested parties, the appearance of the plant is at all times a major consideration. Activated carbon, being a fine black powder and prone to fly into the air at the slightest provocation, is liable to cause an unsightly condition unless carefully handled. It has therefore been realized for some time that the elimination of dust is practically a necessity in water plants. Investigations of this problem demonstrated that the majority of the

dusting occurred when the carbon was emptied into the feeding hopper. In order to overcome dusting at this point a special device (see figure 1) has been developed. This consists of a hopper (A) mounted in a plate (D) so that the top of the feed machine is completely enclosed except at the bottom of the hopper. This hopper is built in such a way that a bag of activated carbon fits snugly when placed in it. The bag, when placed in position, is then cut with the knife edge (C) merely by pulling the handle of the cutting tool (B) A few minutes should be allowed after the bag is emptied for the dust particles in the feed machine to settle. The empty bag should then be removed, and placed in a suitable receptable until finally disposed of. This device is now being made commercially and can be purchased from the manufacturers. It is to be hoped that all manufacturers of dry feed equipment will soon include this or equivalent devices as a standard part of their units.

A group of illustrations of units of the type found useful in feeding activated carbon is provided herewith. It is not implied that no other device may be used. The text of the descriptive data concerning the various units is condensed from information furnished by the manufacturers.

Full information regarding any unit may be obtained through correspondence with the firm producing the equipment.

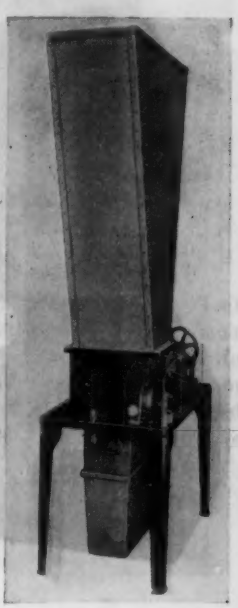


FIG. 2. Bacharach Type RS Dry Carbon Feeder. The hopper of the RS Feeder contains a propelling and cutting device that prevents arching or clogging. Adjustment for any rate of feed can be instantaneously obtained while the feeder is in operation and is continuously maintained with high degree of accuracy and reliability. A specially designed mixing tank makes it possible to use a minimum amount of water and insures an uninterrupted flow of suspended carbon to the point of application. The machine operates noiselessly. No dust can escape from it. It is substantially built, exceedingly simple in construction and all parts are easily accessible.

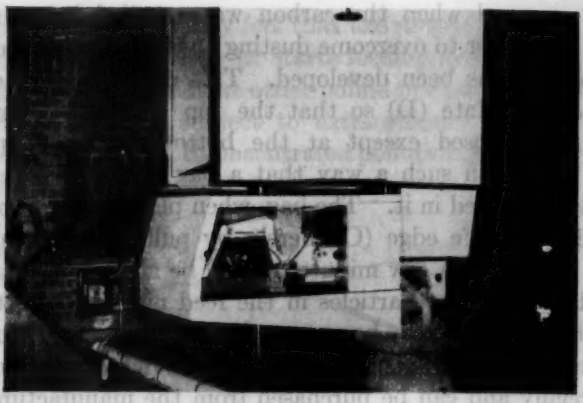


FIG. 3. Jeffrey Dry Feed Machine installed at the Waterworks Park Water Purification Plant at Detroit, Michigan. The Jeffrey Traylor Waytrol is a precision device for delivering by weight a wide variety of materials. It consists of three parts: (1) the vibrating feeder which is totally enclosed; (2) the weigh belt which is enclosed in a steel box having glass sides. This weigh belt is equipped with a calibrated beam which can be set for any given amount; and (3) the control box is the electrical means of regulating the vibrating feeder for discharging to the weigh belt. This control panel can be mounted at any convenient location and lights give visual evidence at all times of correct operation.

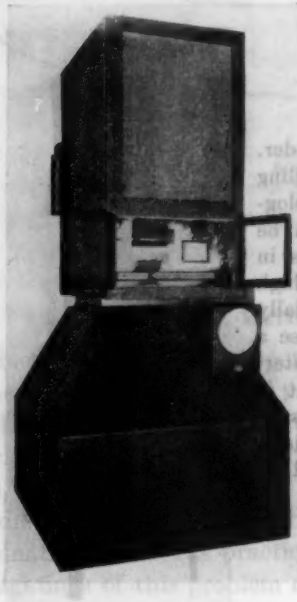


FIG. 4. Omega Dry Feeding Machines to be installed at Cincinnati, Ohio. These are Gravimetric Feeders, controlling the feed by weight and not by volume; mechanical recorder on each feeder; complete dust removal equipment to prevent escape of dust incident to filling the hoppers and any escape of dust from the waste receptacle; a large waste receptacle on wheels to receive the waste cartons; special Omega vibrating agitators on hoppers with rather flat bottom slopes.

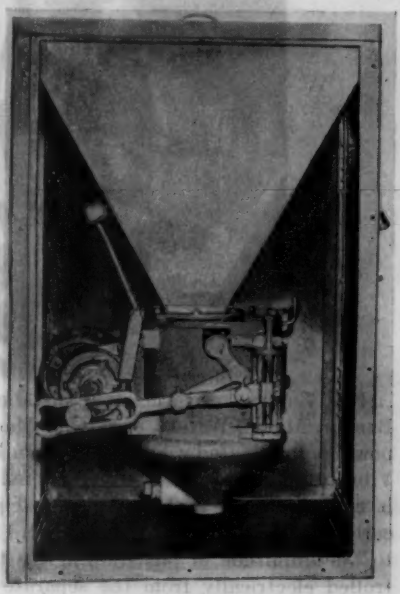


FIG. 5. Permutit Dry Chemical Feed Machine. The Permutit Company's dry feed machine for activated carbon, and other water works chemicals, is of the oscillating plate type. It consists of a storage hopper, an enclosed oscillating feeder driven by a gear head motor and a vortex dissolver, all supported in a rigid steel framework and completely enclosed in a dust proof housing. Positive agitation in the hopper prevents arching. The rate of feed is readily adjustable while the feeder is in operation and the accuracy of feed is within five percent. The feeder is also furnished to feed in proportion to the rate of flow of water through a meter.

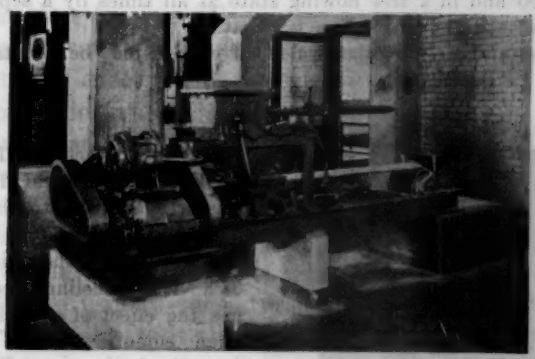


FIG. 6. Poidometer weighing and feeding at the Fridley Filtration Plant of the City of Minneapolis, Minnesota. The poidometer is a precision machine designed to feed bulk material, by weight, in a constant, steady, uniform stream. Its operation is as follows: The scale beam is set at whatever amount of material may be required per foot of belt travel; the gate is then adjusted to suit this weight, and the machine will deliver the predetermined amount of material with accuracy.

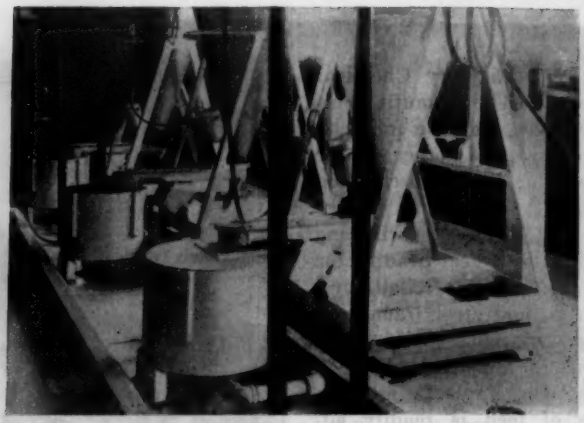


FIG. 7. Dry Feed Machines installed at the new Filtration Plant of the Bangor, Maine, Water Company. Machine in foreground is used for feeding powdered activated carbon. Syntron Dry Feeder Machines are made up of a Vibratory Feeder Conveyor Trough fed from a supply hopper. Each machine is equipped with a separate control panel. The rate of flow of activated carbon through this Vibratory Feeder Conveyor Trough is in direct proportion to the amplitude of pulsation of the trough's electro-magnet, which is controlled electrically from the separate control panel. Hopper contents are kept agitated and in a free flowing state at all times by a separate, small, noiseless, pulsating electro-magnet vibrator.

A dustless and well regulated flow of carbon may be maintained continuously.

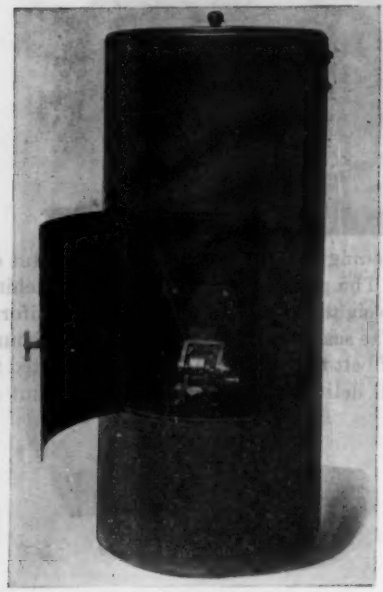


FIG. 8. Wallace and Tiernan Type MOF Dry Chemical Feeder. Particularly adapted to the feeding of activated carbon, the Type MOF Feeder employs the unique principle of a "free wheeling" feed roll which has the effect of extruding a continuous smooth ribbon of chemical from the feed spout. Positive agitation is provided in the feeder hopper which is flexed to prevent arching and to keep each particle of chemical in constant motion. The entire feeder is encased in a dust proof housing to prevent escape of dust into the room or access of dust to the drive mechanism. A wide operating range giving a feed ratio of approximately 100 to 1 makes its capacity adaptable to almost any size plant.

# THE USE OF BENTONITE AS A COAGULANT IN WATER TREATMENT

By H. L. Olin and H. W. Peterson

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Studies made on the dispersion of bentonite, a highly colloidal clay, in turbid waters containing more or less dissolved mineral matter show that under easily controlled conditions coagulation of the clay suspension takes place, to form a readily filtered floc that effectively clears up the turbidity originally present. Moreover the apparent volume of this floc measured by the use of the Imhoff cone is considerably greater than that produced with alum of equal weight and precipitated under optimum conditions. It is further observed that certain bentonites have zeolitic properties i.e., they undergo a base exchange with waters carrying calcium salts and thus produce a definite softening effect. This paper covers the main features of an investigation made on a Utah clay.

Schroeder, Stone and Sanford (1) of the United States Geological Survey define bentonite as a bedded plastic clay which swells greatly on wetting. Ladoo (2) says "the name bentonite has been applied to a group of clay-like minerals characterized by an alkaline or alkaline earth content of five to ten per cent, fine grain size, high adsorptive powers, and usually very strong colloidal properties." Davis and Vacher (3) assign to it the more or less definite composition of the crystalline clay-like minerals montmorillonite and beidellite in both colloidal and crystalline form. Ross and Shannon (4) consider it to be a "rock composed essentially of a crystalline clay-like mineral formed by devitrification and chemical alteration of a glassy, igneous material, usually a tuff or volcanic ash, and often containing accessory crystal grains of feldspar, biotite and quartz." The average composition of 8 bentonites from different sources as shown by their data is given in Table I.

* Contributed record of research.

For the purpose of this discussion bentonite will be defined as a natural hydrous aluminum silicate capable of swelling reversibly in water to at least six times its original apparent volume to form a gelatinous mass with the consistency and appearance of grease. It occurs naturally in strata from a few inches to several feet in thickness which break with a conchoidal fracture. Deposits of bentonite are found in South Dakota, Nevada, New Mexico, Wyoming, California, Utah, Texas, Idaho and in Canada.

## EXPERIMENTAL STUDIES

While directed mainly toward establishing the conditions under which bentonites may be used in clarifying turbid water, this study covered in addition work on their general colloidal and zeolitic properties. The specific clay used was obtained from a deposit near Redmond, Sevier County, Utah, where it occurs as a stratum 12 inches

TABLE I

*Average Composition of Eight Bentonites*

| COMPONENT | PERCENTAGE | COMPONENT | PERCENTAGE |
|---|---|---|---|
| $SiO_2$ | 51.6 | MgO | 4.7 |
| $Al_2O_3$ | 16.7 | $Na_2O$ | 1.2 |
| $Fe_2O_3$ | 2.6 | TeO | 0.7 |
| CaO | 2.4 | $H_2O$ | 21.0 |

thick covered with a foot of conglomerate overburden. After treatment with a saturated solution of sodium chloride no soluble calcium was found in the effluent; whence it was concluded that the specimen was a sodium rather than a calcium bentonite. On immersion in distilled water it swelled to a volume 10.8 times that occupied in its dry state. A five per cent stock suspension was made up in some quantity for use in subsequent work; it remained apparently unchanged for more than six months.

## BASE EXCHANGE PROPERTIES OF BENTONITE

Bentonite has the property of allowing easy exchange of the alkali and the alkaline earth ions and it is possible that the differences in colloidal properties of various clays may be due to variations in composition resulting from such exchanges. According to Mattson (5) a high base exchange capacity which is equivalent to a high osmotic

hydration of micelles, is sufficient to account for stability of a suspension. The amount of exchangeable matter varies in different clays. Jenny (6) studied samples having 2.07 grams of replaceable sodium per 100 grams of clay. Woodman and Taylor (7) found 1.607 grams of sodium and 0.478 gram of calcium and Drossdoff (8), 3.381 grams of sodium in like amounts of other bentonites. It appears however from our own studies that the exchange equilibrium is a function of the acidity or alkalinity of the solution. In Figure 1 appear the results of a series of equilibrium measurements made at different pH values which indicate that below the neutral point the exchange effect

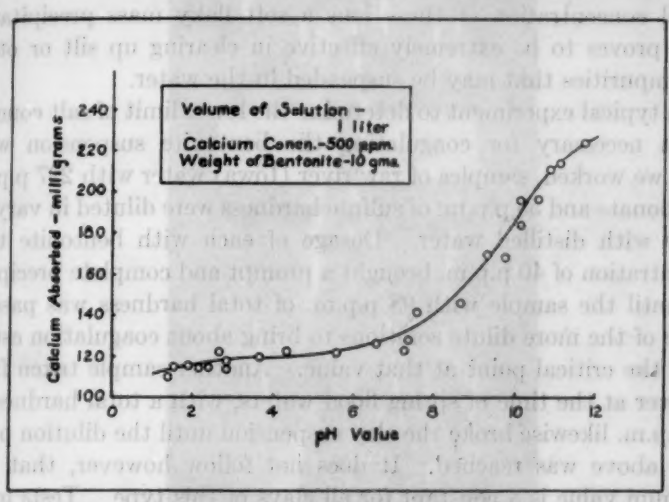


FIG. 1. EFFECT OF pH ON THE ZEOLITIC ACTION OF BENTONITE

is small and that beyond this critical value zeolitic action increases rapidly with mounting alkalinity.

Whatever the possibility of using bentonites as the means for softening hard waters the zeolitic properties of sodium bentonites take on added significance when the clays are to be used as clarifying agents for waters containing carbonate hardness. In standard practice employing the sulfates of iron or aluminum the unstable bicarbonate compounds are converted into the more undesirable sulfates which constitute permanent hardness, whereas the clay actually reduces total hardness however slight that reduction may be. In the alum treatment of the water at Iowa City for example which has about 230 parts of carbonate hardness, raw, the conversion amounts to 20–30 p.p.m.

## BENTONITE AS A FLOCCULATING AGENT

The obvious reactions of certain bentonites when suspended in water readily show them to possess extraordinary colloidal properties. Most conspicuous in the behavior of these clays is the enormous volume increase due to hydration; the formation of a creamy suspension virtually greasy to the touch; and the remarkable permanency of the system in the sol state when maintained under optimum conditions. Like other colloids however, these suspensions are quickly and completely flocculated in contact with solutions of salts or other ionizable substances. As the sol breaks therefore, at the critical concentration of these ions a soft flaky mass precipitates, which proves to be extremely effective in clearing up silt or other solid impurities that may be suspended in the water.

In a typical experiment to determine the lower limit of salt concentration necessary for coagulating the bentonite suspension with which we worked, samples of raw river (Iowa) water with 227 p.p.m. of carbonate and 53 p.p.m. of sulfate hardness were diluted in varying degree with distilled water. Dosage of each with bentonite to a concentration of 40 p.p.m. brought a prompt and complete precipitation until the sample with 98 p.p.m. of total hardness was passed; failure of the more dilute solutions to bring about coagulation established the critical point at that value. Another sample taken from the river at the time of spring flood waters, with a total hardness of 161 p.p.m. likewise broke the clay suspension until the dilution point noted above was reached. It does not follow however, that this minimum value is a constant for all clays of this type. Tests made on a number from other sources indicate that coagulation may occur with salt concentrations as low as one-half the figure quoted above. In the treatment of waters nearly or entirely soft the addition of some reagent preferably lime, is clearly indicated. Studies of this phase of the problem are being carried on at the present time and it appears that distilled water suspensions of bentonite in concentrations varying from zero to 300 may be flocculated and settled at lime concentrations of 50 p.p.m. or less (calculated as $CaCO_3$). Alum may of course also serve as a precipitating agent and some tests on distilled water suspensions of bentonite of concentrations ranging from 20 to 400 p.p.m. showed that coagulation and sedimentation were effected by alum additions in accordance with the linear relationship

$$x = 1.4\,y$$

where x and y are concentrations of alum and bentonite respectively.

### COMPARATIVE VOLUME OF ALUM AND OF BENTONITE FLOCS

Since the volume of the floc resulting from the use of a coagulant in water treatment is directly related to its effectiveness in removing the suspended matter causing turbidity it follows that comparative studies of floc volumes produced by different chemical agents may serve as a means of rating their relative values for use in practice. With this purpose in mind we arranged a series of tests in which volumes of settled floc of alum and of bentonite from equivalent

FIG. 2. COMPARATIVE VOLUMES AND DENSITIES OF EQUIVALENT BENTONITE AND ALUM FLOCS

concentrations (100 p.p.m.) were observed as they settled in adjacent Imhoff cones, and plotted them as functions of time. These cones were filled to the mark with identical samples of water of known pH (7.57) and total hardness (281) and to them were added alum and bentonite respectively to bring the dosage to 100 p.p.m. in each case. In a second and a third set of experiments this was reduced to 40 p.p.m. and 20 p.p.m. in descending order, while temperatures and other factors such as stirring, that might influence rate and amount

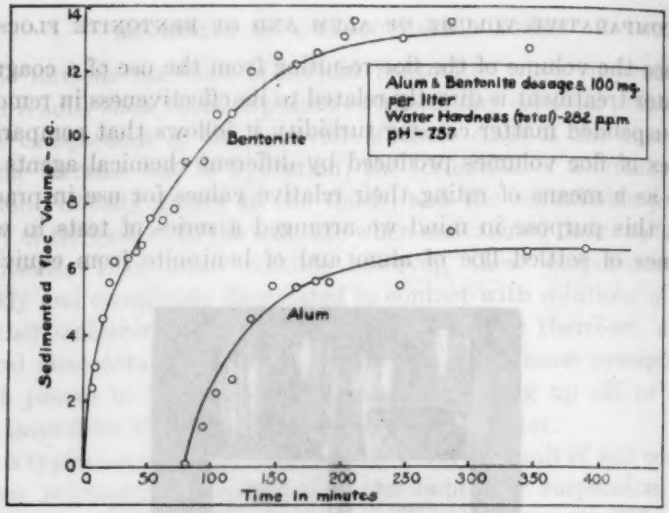


FIG. 3. RELATIVE VOLUMES AND SETTLING RATES OF BENTONITE AND ALUM FLOCS

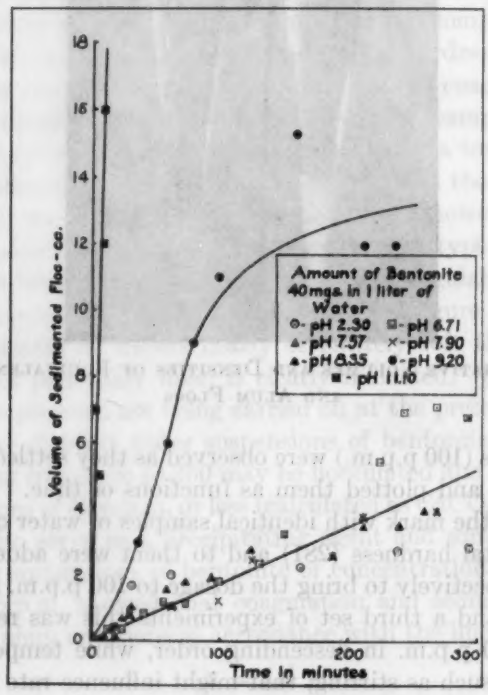


FIG. 4. EFFECT OF pH ON RATE OF BENTONITE SEDIMENTATION

of settling were kept as nearly constant as possible. The relative volumes and densities of these precipitates after sedimentation are indicated by Figure 2.

The results of the tests at the highest concentrations are plotted in Figure 3. It is quite evident that the bentonite not only provides a much greater volume of settled floc but that its rates of reaction and precipitation are much higher than those for alum. Results of measurements at the lower concentrations (not plotted) showed at the end of 6 hours volumes of alum floc of 0.1 cc. and 0.0 cc., while those of bentonite were 4.5 cc. and 3.0 respectively. Indeed at the 40 p.p.m. concentration not until the end of more than 5 hours did any precipitate of alum floc settle at all while at 20 p.p.m. sedimentation was visible only after 24 hours.

### EFFECT OF pH ON VOLUME OF FLOC AND SPEED OF SETTLING

Preliminary studies made on the effect of pH range on the coagulating and settling properties of bentonite suspensions need further study and check. However, results of the work done so far on water samples with 280 p.p.m. of total hardness but varying in pH from 2.3 to 11.1, to which addition of 40 p.p.m. of bentonite were made, are plotted in Figure 4. It appears that until the pH point amounts to values of 9 or more floc volumes and settling rates are fairly constant, even at relatively high acidities. In this respect the clay differs radically from alum which requires close pH control for optimum results.

### TURBIDITY AND FILTRATION STUDIES

In order to test the effectiveness of bentonite in removing turbidity due to suspended silt, samples of raw river water collected under flood conditions, containing 130 parts of carbonate hardness, 31 parts of sulfate, 270 parts of turbidity and with a pH of 7.7, were dosed with the bentonite slurry to bring the clay concentrations to 50, 40, 30, 25, 15 and 10 p.p.m. respectively. In addition a seventh sample was treated with alum to 20 p.p.m. The water under treatment was first shaken vigorously, then stirred gently for 30 minutes, but finally allowed to stand for half an hour longer. Coagulation was evident from the very first in every case where the treatment was adequate, the floc being visible through the tube walls.

At the end of an hour, illumination of the test tubes with the Tyndall beam showed the samples treated with bentonite down to

and including the sample with 15 p.p.m., to be coagulated and almost completely clarified while that with the alum treatment was still turbid. It is interesting to note that the turbidity dosage at the university water plant at the time the sample was taken was 67 parts of alum and 33 of lime.

Filtration of bentonite flocs apparently presents no new or unusual problems. In tests to determine the relative rates of flow of bento-

TABLE II

*Comparative Volumes of Flow, Bentonite and Alum Flocs*

| PERIODS OF 30 SECONDS | FLOW, BENTONITE TREATED WATER | FLOW, ALUM TREATED WATER |
|---|---|---|
| | *cc.* | *cc.* |
| 1 | 266.2 | 252.5 |
| 2 | 266.6 | 253.6 |
| 3 | 266.0 | 254.0 |
| 4 | 265.0 | 255.7 |
| 5 | 262.0 | 253.6 |
| 6 | 262.0 | 254.4 |
| 7 | 263.0 | 253.3 |
| 8 | 262.0 | 254.9 |
| 9 | | 254.2 |

TABLE III

*U. S. Production and Sales of Bentonite*

| YEAR | AMOUNT | VALUE |
|---|---|---|
| | *short tons* | |
| 1931 | 78,815 | $472,045 |
| 1932 | 71,613 | 503,673 |
| 1933 | 117,428 | 760,174 |
| 1934 | 215,339 | 1,050,578 |
| 1935 | 189,840 | 1,071,412 |

nite and alum sludges through sand filters, use was made of laboratory models consisting of glass tubes 2.5 inches in diameter and 40 inches long, filled with sand and gravel to simulate cross sections of the type employed in plant practice. Two samples of raw water of 286 total hardness, turbidity 10 and with a pH of 7.6 were dosed respectively to 100 parts of bentonite and 100 of alum. After complete sedimentation the apparent volume of sludge in the first was

13.0 cc.; in the second, 6.4 cc. After being vigorously stirred the contents of both cones were passed through parallel filters under constant head and the rates of flow of the clear effluents carefully measured. Table II gives the results of these observations.

The free filtering characteristics of bentonite illustrated by the data given above were further verified in numerous observations made later under similar conditions when little change in rate of flow was noted even after a long series of filtration cycles without backwashing.

## PRODUCTION DATA

According to Minerals Year Book for 1936 (9), the price of bentonite, 96 per cent through 300 mesh, f.o.b. points in Wyoming, was constant throughout the year at $17.50 per ton; 96 per cent through 200 mesh, was quoted at $12.50. For its possible use in water treatment however, it is probable that fine grinding would not be necessary since the clay is added in slurry form and the larger lumps slake down readily.

Table III shows the United States production and sales for the five year period (1930–5).

## SUMMARY

It appears from our studies that the sodium bentonites may be used as coagulants in the treatment of waters having a definite minimum concentration of dissolved electrolytes and that the turbidity of waters so treated may be reduced to zero. The coagulative effect moreover is to a high degree independent of pH value, taking place even deep in the acid range. Turbid soft waters require tempering with lime or other reagents. It appears also that with equivalent dosages bentonite flocs are uniformly more voluminous and more rapid in sedimentation than those produced with alum.

The zeolitic properties of bentonite while not of major importance nevertheless give the clay a definite advantage over alum in the treatment of waters of high carbonate hardness since no conversion into the more troublesome sulfate hardness takes place.

Further investigations in this field of study are being made.

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# SERVICE LEAKAGES—CAUSES AND CONTROL*

By Felix Seligman

(*Manager, Water and Light Department, Duluth, Minn.*)

Every locality is confronted with problems of one kind or another. In Duluth, which has often been vividly described as being one mile high and twenty-five miles long, we are faced with problems having considerable bearing upon the operation of a water plant. The topography has made necessary the filling in of many ravines, and of creeks and rivers which at one time found their way into Lake Superior through areas now laid out in streets. The large amount of rock of the hardest kind found throughout the entire city limits makes the installation of services in most instances difficult and very expensive; and, where rock is not encountered, hard pan and clay are the materials with which we have to contend. The severe winters necessitate the laying of services in trenches at least seven feet in depth. The fills which have been made in the old river and creek beds offer their problems. So this account of our experience with service leakages, their causes and control—just one phase of our problems—must take into consideration the conditions encountered; and, because of these varying conditions, our experiences with the problem of installing and maintaining our water system should cover the usual conditions encountered throughout the State.

Generally speaking, the causes of service leaks can be divided into three groups. (1) Controllable, (2) Partly Controllable, and (3) Not Controllable.

*Group 1. Controllable.* In this first group may be classed the regulations governing the methods under which the services are installed, the material of which the service is constructed, the workmanship, and the inspection of the completed service.

*Group 2. Partly Controllable.* In group 2 may be found such causes as frost action, excavation in or near the vicinity of a service, electrolysis and galvanic action, corrosion, the

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location and method of installing stop boxes, expansion and contraction, movement of either the dwelling supplied or the main from which the service is supplied, and defective material.

*Group 3. Not Controllable.* In group 3 we may class worn-out services due to many years of usage.

It is interesting to note that in classifying the causes of leakages the majority are either controllable or partly so.

## GROUP 1

If future trouble from leakages in services is to be avoided, too much emphasis cannot be placed upon those factors which are within the control of the utility. Regulations should be adopted which stress the importance of the material used, good workmanship, and rigid inspections.

The material used in the construction of services should be of the best that can be obtained under the present development of the arts. Superintendents of water plants should always be open-minded and willing to consider material which is better than that available at the present time.

But the best of material will be of little avail if it is improperly installed. Good workmanship pays big dividends, both in controlling the unaccounted-for water and in avoiding the ill-will which results from the continual tearing up of streets and lawns. Good workmanship, therefore, must be insisted upon in all stages of the installation. It is always advisable to have the completed job thoroughly inspected by one who is familiar with all phases of the work and conscious of the importance of having services properly installed. Material, workmanship and inspections are matters that can be incorporated into the regulations with the objective of getting the best possible service obtainable.

## GROUP 2

*Frost Action:* In looking over Group 2 and the order in which the causes classified as "partly controllable" have been stated, we find "frost action" first mentioned. Frost action has been classified as partly controllable insofar as, generally speaking, services should be laid below the greatest frost depth usually encountered. However, occasions will arise, such as arose during the past year, where records are broken and frost penetrates to a depth beyond that which had been previously recorded.

*Excavation in or near the Vicinity of a Service:* While in many cases it is not controllable by the Water department, excavation in or near the vicinity of a service should be watched very closely by employees as proper supervision at the right time will go a long way in preventing trouble in the future.

*Electrolysis and Galvanic Action:* Electrolysis and galvanic action are both factors to which the general waterworks personnel have paid little or no attention. They are, and will continue to be, responsible for numerous failures in water services. However, they are partially controllable in that the installation of an insulator between the service and the main will remove most of the trouble experienced from man-made stray currents. Galvanic and stray current electrolysis from natural causes, while they exist in certain localities, are not so destructive in their action and, generally speaking, can be ignored.

*Corrosion:* Corrosion could well be classified as a controllable factor in that if the service is constructed of the proper material there is no trouble from this source. However, certain localities have soils which are conducive to a rapid rate of corrosion of almost any material, or have waters which, in themselves, cause very rapid deterioration of some materials.

*Location and Method of Installing Stop Boxes:* The location and method of installing stop boxes should be closely supervised to see that they are properly installed and that the location is one which will not be continually disturbed by the action of heavy equipment passing over the same.

*Expansion and Contraction:* Failures due to expansion and contraction can be partially avoided by the proper placing of the service in the trench.

*Movement of Dwelling or Main:* The effect of breakage or leakage due to the movement of a dwelling or main can also be compensated for by the looping of service at these locations so as to permit movement without breakage of pipe.

*Defective Material:* Trouble from defective material can be partly avoided by purchasing only the best of material, and by inspecting it before installation in the ground.

GROUP 3

*Worn-Out Services:* Worn-out services are the one cause of leakage which I have classed as "not controllable." As a matter of fact, this source could be controllable if services that have been in the ground

for forty to fifty years were renewed. Were we to classify the causes of the majority of leakages in services, we would undoubtedly find the old "wiped joint" the worst offender. With this type of joint, regardless of its usually perfect appearance, one never was sure of its dependability. The Department which I represent has always employed plumbers who were considered experts in the wiping of these joints; but, in spite of this precaution, the majority of our failures have occurred at the wiped joint. The material available today does not require the wiped joint, but can be connected to fittings by the average workman, who, after a short period of instruction, can make the joint as efficiently as an expert.

## METHODS IN DULUTH

I am going to give a brief description of the methods employed in Duluth in the laying and maintenance of street and house services. Should anyone desire more detailed information, a copy of our Rules and Regulations can be obtained.

For convenience, we divide services into two classes:

(1) Street service:—that part of the service from the main up to and including the stop box and cock, which is usually placed near or at the curb.

(2) The house service:—that part of the service from the curb cock to and including the building, up to and including the meter.

The street service is installed solely and exclusively by the department, by its own regular employees and equipment. There is, therefore, no other inspection than the foreman's supervision of the installation. Because the department furnishes only such material as is specified by the rules and regulations, the work is of the best obtainable under the present known methods.

Only licensed plumbers, working under a license issued by the department, may install the house service, thus enabling the department to keep close supervision over all plumbers doing this work. Although the plumber furnishes all of the material and labor which goes into the house service, the department inspects the work during and after construction to see that the workmanship as well as the material used are of the best and in good condition. By this procedure the department protects itself as well as its customer.

For many years lead services were installed, but during recent years the rules and regulations have been revised to permit the

installation of only copper pipe for water services. The smallest size permitted is ¾ inches. Our experience with copper pipe has been highly satisfactory. It is not subject to corrosion, is lighter in weight, and much easier to handle and install. The bore of the pipe does not scale, but is smooth and very uniform.

When a water service is to be installed in a rock trench, our regulations specify that the trench must be 3 inches deeper than that required for an earth trench; that 3 inches of earth must first be placed in the bottom of the rock trench before the pipe is laid, and that care must be exercised to see that there is not less than 12 inches of dirt placed over the service pipe before any rock, etc., is thrown back into the trench. Large stones or boulders must not be placed in the trench after services have been laid. The pipe must be laid "wavy," not absolutely straight; and a good "wave" or small loop must be provided next to the curb and also below the basement floor at the point where the service is to come through. Placing the pipe in the trench as specified not only minimizes the probability of any breakage, leakage, or the stripping of the service pipe from the curb cock, but also permits contraction and expansion and eliminates stresses that would otherwise occur if this procedure were not followed.

The department has found strict supervision very necessary in the installation of services, because, regardless of the type of plumber and laborers employed, carelessness is so apt to creep into the work—the service pipe may not be installed to the depth required, or it may be laid in the bottom of a rock trench, or large stones and rough material may be thrown down against it when the trench is back-filled.

It has been our experience that one service should supply only one building, and that each service should be laid throughout the entire length on or in the land of the building which the service supplies; for neighborhood quarrels are common where one service supplies more than one dwelling, or where the service lines are laid in adjoining property to the dwelling supplied. In the early years of the department's activities, the regulations were not always enforced to the letter; as a consequence, we have a number of the older dwellings supplied from one service. Today, these services have outlived their period of usefulness, but the problem of who is to pay for the renewing of the service is one that entails a lot of trouble, both for the property owners and for the department.

Leaks are found mainly by sound, by listening with ear placed against the pipes in the building, or by placing the street key in the

curb box and listening with ear against the key. Water escaping under heavy pressure makes a decided humming or singing sound easily recognized by experienced water men. We cannot use the meters to test for leaks because the meters are set in the buildings. Very long services should be metered at the curb, for under this set-up the meter will record any leaks in the service line. To protect itself, the department insists upon having long services metered at the curb. Sometimes the sewer is used in determining leaks when there is no sound or appearance of water above the ground. In such cases, an investigation of the sewer manhole, if available, will help determine if there is a leak. At other times we tap the sewer for the purpose of determining whether a leak is present.

## CONCLUSION

Appreciating the value of trouble-free services, our department has from time to time revised its regulations whenever the swiftly moving pace of industrial progress has made revisions desirable as well as necessary; it has always been ready to change its methods when experience has demonstrated a better way. Since it revised the regulations to take advantage of the improved materials available for the construction of services, it has never had a failure of the new material. I am, therefore, firmly convinced that, if constructed of the best of materials and installed under strict supervision, services should last practically indefinitely without giving trouble of any kind.

# METER MAINTENANCE AND REPAIR*

By George A. Roden

(*Superintendent, Meter Division, St. Paul*)

Upon complaint originating with meter reader or consumer, meters are removed and brought to shop for test and the necessary repairs. Each meter is tested and the results are recorded. Meters found stopped are dismantled and the reason for stoppage determined. If occasioned by consumer's neglect through freezing or burning by hot water, these circumstances are noted. Any condition having a bearing on consumption is carefully considered and results included in information furnished the accounting division.

After test, meters are dismantled and the parts cleaned with acid wash. Registers are repaired by a mechanic specializing in such work. It has been found in the past that men repairing meters are prone to neglect the registers; such indifference is avoided by removing the register and having one man committed to such work. Furthermore, we have the assurance that meters leave the shop with a zero reading. Extensive repairs to intermediate gears are also made by one man or under his supervision. Thus, the process of repairing a meter resolves itself into fitting the disc into the chamber. Where the chamber is worn at top or bottom, the disc is shimmed under the half balls and ground to a fit, in much the same manner as an auto valve. This fitting of the disc to the chamber is the most important feature of the repair; when this is satisfactorily accomplished, the balance of the repairs is merely a matter of assembling. After assembly, meters are tested by the individual repairman and then placed in readiness for painting. Certain percentages are required and it is strictly forbidden to accomplish satisfactory registration through the substitution of change gears varying by more than two teeth. Meters are re-tested at frequent intervals by foreman to prevent slipshod work. After a paint spray, meters are tagged and ready for circulation.

The repairs of meters three inches or over are taken care of by one man who has been trained in such work. The testing of a large meter

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consists of rating flows from the smallest to the largest orifice by graduated steps. Especially is this true of the compound meters where particular attention is paid to the "change-over period." Such a test will indicate the condition of the compound valve. The by-pass meter is tested individually. The large flows will indicate the accuracy of the main meter. Where the compound meter meets the specified requirements of these three phases of test, the repairs have been satisfactory.

No field tests are ever made on small meters. Such a test requires too much handling of plumbing, and where this is in a poor condition, much grief results. The field test on large meters is the exception rather than the rule. We depend on a periodic removal and test to maintain our meters functioning properly. Such intervals are as follows: Three, four and six inch meters every five, four and three years respectively. On the small meters, we have as yet not decided on any definite interval.

Over a period of a year the following reasons for removing meters have been compiled:

| *Reasons* | *per cent* | *Totals* |
|---|---|---|
| Stopped: | | |
| Burned | 7 | 396 |
| Frozen | 6 | 305 |
| Meter defective | 13 | 700 |
| Sand | 2 | 81 |
| Register defective | 22 | 1154 |
| Leaking | 29 | 1529 |
| S.O.R. | 10 | 511 |
| Test | | 11 |
| Meter disconnected | 1 | 73 |
| Plumbing change | 3 | 122 |
| Noisy | 1 | 66 |
| P.T. | 2 | 84 |
| Pressure | | 16 |
| Not justified | 4 | 181 |
| On backwards | | 1 |
| Total | | 5230 |

Explanation:

"S.O.R."—Service shut off and meter removed.

"Test"—Removed because of high bill complaint, investigation or request for test by consumer.

"Plumbing Change"—Meters changed because of plumbing or change in meter size.

"Meters Disconnected"—Meters set out by plumbers to drain system.

Of the above, the item of leaking is the cause for most concern. The greater part of such complaints is covered under the heading of spindle leaks. At one time we were under the impression that we alone had such trouble, but experience has proved that this is a defect in many if not all meters. At this time it might be appropriate to express a desire for a "meter clinic," in which the views and experiences of meter users could be aired and related. Certainly, meters are important enough to warrant a great deal of attention and subsequent improvement.

We may have a bounteous source of supply, an effective purification plant, an efficient distribution system, but a trustworthy meter guards the revenue.

# PUBLIC RELATIONS*

By J. C. Flanagan

(*Assistant Secretary and Registrar, St. Paul, Minnesota.*)

The subject of public relations is a vital matter with water utility service, especially with water departments that are municipally operated. For the purpose of this paper, I shall refer a great deal to the practices in force in the City of Saint Paul system, particularly to the rules and regulations adopted as council ordinances, which embrace all the features incident to a public water supply system.

The water department of the city of Saint Paul, by a legislative act fifty-four years ago, became a municipal activity. The provisions governing the department are set up by both legislative acts and city ordinances. The department must operate on its own resources. It must retire all its own bonds and all other financial obligations, and it must do this as well as pay all its operating and maintenance costs solely from its revenue and income. No part of money raised by taxation for the government of the city of St. Paul has been or can be used for the conduct of its water department, nor can the water department moneys be used for functions other than for its own requirements.

In every public water supply system there is a Director of Public Relations, under the title of General Superintendent, Engineer, Secretary, Registrar, or what not, who has the final contact with the patrons of the department, especially in the matter of demands for additional service and disposition of various complaints. Complaints of a similar nature are experienced by all water service officials and perhaps it might be well to review some of the causes which disturb the patrons more than others and which of course necessarily require careful handling so that the complainant may be convinced of the justice of the water service system's position.

High bills are a continual source of complaint because too often they are the result of wastage due to leaks in the service pipe in the premises, flushing of toilets and dripping at taps, etc., neglect in most

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part by the tenant's failure to inform the owner, or the owner's neglect in the matter of repairs.

Occasionally, high bills are the result of underground leaks where the service pipe running into an adjoining building or underground for a short distance in the premises before it rises, makes the existence of leaks harder to detect until revealed through the meter reading.

Then there are the increases due to seasonal demands, particularly evident in Saint Paul during the summer of 1936 when for a period of sixty-six days and nights there was no rainfall in the city, causing a heavily increased demand for irrigating lawns, gardens, shrubs, etc. Too often patrons fail to understand or realize the quantity of water which may be delivered through the size of meter established on their premises, and while they may admit that they used a little more water, they overlook the fact that their generosity under those circumstances went beyond their realization of the quantity which was so used. Therefore it becomes necessary in such cases to explain clearly to that patron that, based on the pressure at his place, plus distance carried, the amount of water that may be delivered every minute through the size of meter at his premises, and computing that into hours, it then becomes a matter of simple arithmetic to determine that the excess charge is reasonable and due to his own act.

Penalties when imposed because of failure to pay the bill at the due date, cause many complaints. It is the impression in the minds of some people that a check placed in the mail on the penalty due date which comes in the following day or later is considered as payment within the prescribed time. That would be true did we not state that the payment must be received at the department office on a specified day, and of course there are other objections, such as the patron's absence from the city, claim that he failed to receive the bill, etc., none of which are recognizable under our rules. He must be shown that in conducting any business, and especially a municipal activity, there must necessarily be adequate rules provided that will not permit of discrimination.

During drought periods or hot summer months, there are many complaints of lack of pressure, which is the natural result of increased demand for irrigation purposes coupled with increased domestic use, in which case it must be clearly explained to the patrons that they cannot have both increased demand and maintained pressures at the same time and that the extra demand for irrigation again cuts down the pressure at the premises.

In St. Paul, upon application by the owner, the service pipe from the main in the street to the property line is installed by the department and paid for by the property served, with a guarantee of thirty years by the department against any repairs that may become necessary thereafter, after which, however, repairs are made by the department, and billed against the premises. The department makes no attempt to go into the premises to make repairs, that being the function of a licensed plumber. The water department of the city of Saint Paul owns all meters, charges no rental for them, but requires the property owner to protect the meter against its loss or damage caused by frost or hot water. In cases of the existence of these conditions, it is necessary to make repairs and bill the property for the cost. The greatest objection in regard to these repairs arises from charges made against the property owner because of damage to the meter by hot water. He does not fully appreciate that hot water alone can cause such damage and he seems never to be conscious of the time when it occurred. It requires quite a bit of tact to handle satisfactorily a complaint of this type.

Again there are patrons who, finding their water bills a little higher than the average, immediately conclude that the meter is running fast and request that we change the meter. Well, as every water department manager and others having to do with that branch of service know, instances of water meters running fast are very rare and were a department to comply with all requests of that nature it would require a small army of men to do the work. It is true that water departments should not leave meters continuously on premises more than four or five years because experience proves that meters then show a substantial slowing down, thus depriving the department of considerable revenue.

In the set up in St. Paul the water rate is made up of two parts: (1) a demand (or readiness to serve) charge based on the size of the meter; and (2), the consumption charge, the two making up the full rate. Many people object to the so-called "service charge" and confuse it with a provision to render special services free. They fail to realize that there is a vast difference between a man contracting for a $\frac{5}{8}$-inch or $\frac{3}{4}$-inch service and another demanding a 6-inch supply. When the normal user contracts for a supply through a $\frac{3}{4}$-inch pipe, he is entitled to the use of all the water that will pass through that size pipe. He may use it only a small percentage of the time, but the water department is under contract to furnish him

water 100 percent of the time. The liability of the department extends as fully to the larger consumer as to the smaller one, with the essential distinction that the larger the service the greater the liability. Therefore, the writer believes that a water rate in order to do justice to every consumer must be broken up into two parts:

A—Readiness to serve, or demand, charge;

B—Consumption, or delivery, charge.

There are many other sources of complaints which the writer regards as minor and easily adjusted and, therefore, not important enough to be itemized in this paper.

Too often, patrons when registering objections to the restrictions or regulations of the department are dismissed rather curtly with the response, "That is the rule of the department," which he fully realizes. It would be better if he be shown that not because the rules prevent compliance with his request but why the rules are in effect when he will realize that no injustice is being done him.

Having in mind the foregoing, the writer suggests in the matter of public relations the contacting officers and employees should observe the following practice:

1st—Take no personal offense;

2nd—Listen to the consumer; let him do the talking;

3rd—Go to the bottom of his problem and settle it before he leaves, if at all possible;

4th—Keep sweet at all times.

By these observances they will succeed in establishing nearly 100 percent cordial relations with the patrons of the water department.

# ECONOMICAL SIZES FOR WATER FORCE MAINS*

By Albert O. True

(*Sanitary Engineer, Proximity Mfg. Co., Greensboro, N. C.*)

Research and observations made in the last few years on the friction heads for iron pipes conveying water indicate that the increase in this friction with age is considerably more rapid than has been commonly supposed, and as recorded in many of the pipe tables now in use.

The significance of these investigations, as affecting pumping and pipe line economy, is emphasized in a recent brief article by Mr. F. A. Barbour, Consulting Engineer, and a Past President of the A. W. W.A., under the title "Economic Significance of Improved Pipe Linings."[1] This article indicates a realization by some of the profession that there has been a considerable loss of economy in the annual cost of pumping and distributing water resulting from a too favorable estimate of the probable value of C in the Chezy formula following a few years use of the pipe. This condition will presumably be largely corrected if, and when, pipes of other material than iron, or iron pipes with smooth linings, in which a permanently high value of C obtains, become highly economic and practical.

The writer has recently examined a number of cast iron tar dipped pipe lines which have been in service from 10 to 15 years which are not delivering water to the point of consumption with economy owing to the fact that the coefficient C has fallen off more rapidly than was anticipated. In some of these lines the velocities are too high for economy even if it had been possible to maintain a coefficient as high as 100. Too many systems have been designed, where pumping is required, without proper reference to the relative costs of pumping and the interest on investment. This results many times in too high velocity in the pipe for economy.

The annual cost of a pipe line to transport pumped water consists

* Developed from a paper presented before the North Carolina section, October, 1936.

[1] Jour. A. W. W. A., **28**: 9, 1344 (1936).

of the two items of the annual interest and upkeep of the line plus the cost of power for pumping. Each is proportional to the length of the line. If the line be too small diameter, the velocity and resulting friction will be high and the increased cost of pumping will render the layout uneconomical. If a diameter of pipe very much too large is selected, the interest on the cost of the installation will be excessive, also rendering the design uneconomical. The most economical size of pipe for the service required lies somewhere between these two sizes. The total annual cost for this size pipe will be a minimum, and the velocity obtaining will be the economical velocity.

In order to arrive at this economical velocity in any particular case, several local cost factors must be known, and applied with good engineering judgment with reference to local conditions and the service to be performed. All of these factors can be applied equally well in arriving at the most economical velocity in pipes or coatings of any material provided some knowledge is available of the life of the material to be used and the probable depreciation, if any, of the friction coefficient of the interior pipe surface.

As iron and steel are the most common materials now in use for pipe lines, the following tables of economical velocities and discharge are based on these materials installed and operated under average conditions.

In connection with the tables to follow, the following symbols will be used:

K = the cost of pipe laid in cents per foot.
i = the cost of the metal in pipe in cents per pound.
x = the diameter of the pipe in inches.
V = the velocity in the pipe in feet per second.
s = the loss of head in pipe from friction in feet per foot of length.
Q = the quantity of water pumped continuously in cubic feet per second.
r = the rate of interest expressed in dollars and taken at $0.06.
C = the friction factor in the well known Chezy formula $V = C\sqrt{Rs}$.
p = the annual cost of pumping 1 cubic foot per second continuously against 1 foot ahead, expressed in cents.
y = the annual cost of the installation (total annual cost), expressed in cents.

As these quantities are variables, whose values will depend on

when and where they may be used, something should be said on the range of these values, and the data from which they may be derived with reasonable accuracy. The values used in the following tables are assumed as approximating average figures, and should not be used as applying to cases in which they diverge greatly from such averages because of local or other considerations.

K is the total first cost per linear foot of the completed pipe. This is the first cost of the installation, and will depend on many factors as the size and material of the pipe, freight and haul, method of trenching—by hand or machine, rates for labor, percentage of rock to be trenched, kind of pipe joints employed, amount of pavement to be cut and replaced, unusually difficult working conditions in urban

TABLE 1

*Cost of Pipe per Foot Laid, in Cents*

| WROUGHT IRON AND STEEL | | CAST IRON | | | |
|---|---|---|---|---|---|
| ⅛″ | 22 | 4″ | 85 | 24″ | 770 |
| ¼″ | 23 | 6″ | 120 | 30″ | 1,071 |
| ⅜″ | 24 | 8″ | 165 | 36″ | 1,427 |
| ½″ | 25 | 10″ | 213 | 42″ | 1,875 |
| ¾″ | 28 | 12″ | 267 | 48″ | 2,351 |
| 1″ | 31 | 14″ | 333 | 54″ | 2,732 |
| 1¼″ | 37 | 16″ | 408 | 60″ | 2,985 |
| 1½″ | 41 | 18″ | 489 | 72″ | 4,185 |
| 2″ | 50 | 20″ | 587 | 84″ | 5,300 |
| 2½″ | 62 | | | | |
| 3″ | 75 | | | | |

localities, depth of cover, etc. In the diagrams, here presented, it has been computed from the formula $K = 20 + 2ix^{1.57}$ in which i has been taken at 5 cents per pound for wrought iron and steel pipe and 2½ cents per pound for Class B cast iron pipe. Cover has been taken at 4 feet over the top of the pipe, in average digging by hand, and no rock to be removed. To illustrate: the cost of 12″ pipe line by the above formula would be

$$\begin{aligned} K &= 5 \times 12^{1.57} + 20 \\ &= 247 + 20 \\ &= 267 \text{ cents or } \$2.67 \text{ per foot.} \end{aligned}$$

This average cost per foot for bell and spigot C.I. pipe would be distributed about as follows:

| | | per cent |
|---|---|---|
| Cost of pipe @ $50.00 per ton | $2.060 | 77.19 |
| Yarn | .008 | .3 |
| Lead | .180 | 6.75 |
| Hauling | .060 | 2.25 |
| Trenching and backfilling | .300 | 11.25 |
| Laying, cutting and caulking | .060 | 2.25 |
| Total | 2.668 | 99.99 |

Similarly for a 48″ pipe:

K = 2200 cents, $22.00 per foot.

| | |
|---|---|
| Cost of pipe | $18.980 |
| Yarn | .025 |
| Lead | .561 |
| Hauling | .351 |
| Trench and backfill | 1.400 |
| Laying, cutting, caulking | .700 |
| | $22.017 |

The cost of metal in the pipe will vary considerably with the market and the location of the point of delivery, and, as this is usually the largest item of cost, due allowance must be made for these fluctuations.

The diameter of the pipe, x, needs little comment. It is, of course, the bore of the pipe, and, in the case of wrought iron and steel pipe of small sizes is the actual diameter in inches, and not "nominal size" as commonly used to designate such pipe.

V is the average velocity in the pipe under flow as determined by the equation

$$V = \frac{Q}{A}$$

in which A is the area of the cross section of the bore in square feet.

s is the slope of the hydraulic grade line along the pipe under flow. It represents the friction loss measured in feet for each linear foot of the pipe line. It is a direct index of non-useful work or wasted power.

Q, the discharge of the pipe line, is self-explanatory. By the use of any hydraulic tables of pipe flow it can be converted into other units readily such as gallons per day or per minute. Conversion between several units can also be quickly obtained, with reasonable accuracy, from figure 3, shown herewith from 0 to 10,000,000 gallons per day. From this it is readily seen that corresponding values in different units

lie along any radius. The inner circle gives the theoretical horse power required to lift a flow of water found on any radius, per foot of lift at 60 percent efficiency of the pumping unit. This, of course, includes no hydraulic friction, being lift only.

The rate of interest, r, is important as it represents the cost of money, or overhead charges on the investment. It also will vary somewhat with time and place and other considerations.

The friction factor, C, is a somewhat elusive variable. Even under the same friction surface at the inner wall of the pipe it will vary slightly for different hydraulic slopes. Many formulae have been devised involving velocity and friction factor none of which give results agreeing with the actual friction under all conditions. Well laid pipe lines, when new, may show values of C as high as 140. Probably few cast iron lines would show as high a coefficient as 140 even under favorable conditions owing to the effect of valves, fittings and bends in the line which increase the friction losses over the theoretic ideal of the perfectly straight pipe. Many cast iron, tar dipped pipe lines are showing, under test, values of C of 80 or even worse after a use of not over 10 years. This disappointing tendency is stimulating the search for better materials or linings with the permanence of cast iron combined with smoothness of well known but less durable materials. Many cities are having pipe lines cleaned periodically by special cleaning machines. Pipes thus cleaned will, in many cases, have a friction coefficient equal to new pipe; but to keep them in this condition will probably require a cleaning every three or four years.

p represents the annual cost of lifting water one foot against friction at the rate of one cubic foot per second on the assumption that the operation is continuous. In the data presented herewith it is calculated at one cent per K. W. H. under an overall efficiency of the pumping set of 80 percent. When so calculated the annual cost is $9.30 which, in the formulae is expressed in cents.

The total yearly cost of the installation and operation of the force main is expressed as the variable y. This includes the annual interest on the first cost of the line, and the annual cost of operating and maintaining the line. While it is not, perhaps, properly a part of this discussion, attention is here called to the fact that the cost of pumping equipment may be considerably affected by the hydraulic conditions in the force main serving the pumps. If the velocity in the force main is excessive, not only will more power be used in over-

coming the resulting friction, but on account of the higher pressures to be handled, heavier and more expensive pumping sets and motors will have to be installed. Thus an additional first cost charge is pyramided on the power cost.

The annual interest cost of the installation of the pipe line plus the annual cost of pumping against friction, per foot of pipe, represented by y will be

$$y = psQ + Kr \qquad (1)$$

The hydraulic slope, or head lost in friction per foot of pipe, will, of course, vary with the age and condition of the pipe wall as determined by the coefficient C in the Chezy formula.

The following values of s in terms of quantity Q and the diameter of pipe x were obtained from the following formulas:

$$s = 198.5 \frac{Q^{1.75}}{x^{4.75}} \quad \text{corresponds to } C = 80 \qquad (a)$$

$$s = 136 \frac{Q^{1.75}}{x^{4.75}} \quad \text{corresponds to } C = 100 \qquad (b)$$

$$s = 100 \frac{Q^{1.75}}{x^{4.75}} \quad \text{corresponds to } C = 120 \qquad (c)$$

$$s = 73.5 \frac{Q^{1.75}}{x^{4.75}} \quad \text{corresponds to } C = 140 \qquad (d)$$

The value of s as given in any one of the formulas (a), (b), (c) or (d) can be substituted in formula (1) to obtain a value of y representing the total yearly cost of the installation per foot of pipe. As all four values, or any such values, would be treated in the same way, only one will be reproduced here for illustration. Consider the value of s in (b) corresponding closely to C = 100. Substituting this value of s in equation (1), the total yearly cost becomes

$$y = 136p \frac{Q^{2.75}}{x^{4.75}} + 20r + 2irx^{1.57} \qquad (2)$$

$$y = 136pQ^{2.75} x^{-4.75} + 20r + 2irx^{1.57} \qquad (3)$$

From inspection of equation (2) it is evident that when x the diameter of the pipe is 0, y, the total cost, is infinite. It therefore follows

that the minimum value of y obtains when the differential of y with respect to x is equal to 0.

Differentiating equation (3)

$$dy = 136pQ^{2.75}(-4.75)x^{-5.75}dx + 0 + 2ir1.\ 57x^{.57}dx$$

and

$$\frac{dy}{dx} = -646pQ^{2.75}\,x^{-5.75} + 3.14\,irx^{.57} = 0$$

solving the above equation for x

$$x = \left(\frac{646pQ^{2.75}}{3.14ir}\right)^{.158}$$

or

$$x = 2.32\left(\frac{p}{ir}\right)^{.158} Q^{.43} \qquad (4)$$

The value of x in (4) is the diameter of pipe for which the total annual cost would be a minimum.

This can also be expressed in terms of the most economical velocity for any size pipe by substituting in equation (4) the values of Q in terms of V and x.

$$Q = .00546Vx^2$$

Substituting in equation (4)

$$x = 2.32\left(\frac{p}{ir}\right)^{.158}(.00546Vx^2)^{.43}$$

$$V^{.43} = \frac{x}{2.32\left(\frac{p}{ir}\right)^{.158}.1063x^{.86}}$$

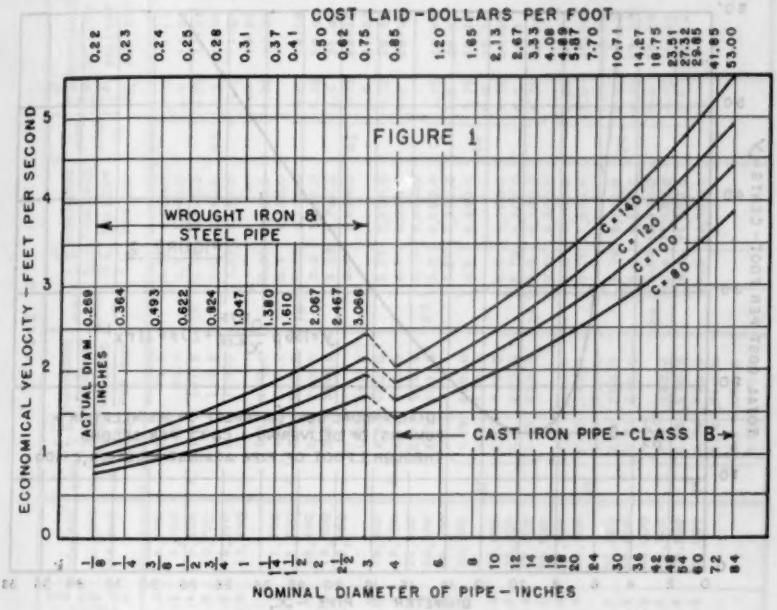
$$V = \left\{4.05\left(\frac{ir}{p}\right)^{.158} x^{.14}\right\}^{2.33}$$

$$V = 26\left(\frac{ir}{p}\right)^{.37} x^{.33} \qquad (5)$$

V in equation (5) is the most economical velocity for a pipe of diameter x when the friction coefficient C is at or near 100.

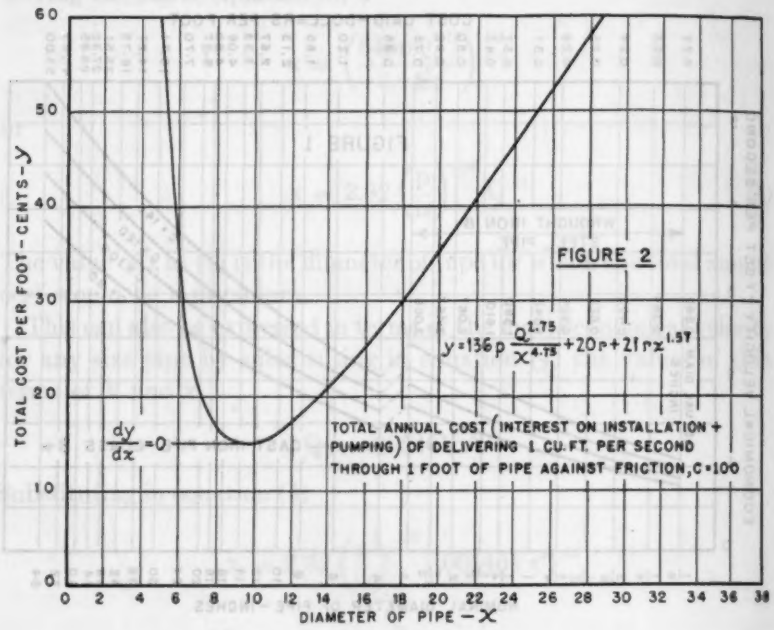
Figure 1 gives the economical velocities in pipes in feet per second for corresponding values of C as derived from equation 5 for the four values of C shown. Also, at the top, the cost of the pipe laid in dollars per foot. The drop in economical velocity from 3″ pipe to 4″ pipe is occasioned by the fact that the cost of metal in the wrought iron or steel pipes is figured at 5 cents per lb., while the cast iron is figured at 2.5 cents per lb.

As already stated, the values given are based on the assumption that pumping is continuous for the year at the rate stated.




Two important considerations are necessary in using the figures as shown: (1) proper engineering allowances must of course be made for future increase in water consumption which will affect the value of Q; (2) as close an estimate as possible must be made of the number of days per year the pumps will be in operation. For example, if the total year's pumping aggregates only 3 months the values of V should be increased by multiplying the table values by $4^{.37}$ or 1.67. If only 6 months multiply by 1.29. 8 months by 1.16, etc. Other factors by which the economical velocities should be multiplied can be taken from the graph on figure 4.

The graph shown in figure 2 is the typical curve of equation (2). This is plotted for a quantity of 1 cubic foot per second, the pipe being 1 foot long and the value of C = 100 for continuous pumping. Diameters of pipe x are plotted as abscissae, and the total annual cost y as ordinates. This particular graph indicates that the most economical diameter of pipe for the conditions stated above would be 9″. As this is an odd size, not less than a 10″ pipe would be used. From the shape of this curve it is readily seen that it is better to err on the side giving a diameter larger than the most economical rather


TOTAL COST PER FOOT - CENTS - y
DIAMETER OF PIPE - X
FIGURE 2
$y = 136p\frac{Q^{1.75}}{X^{4.75}} + 20p + 21px^{1.57}$
$\frac{dy}{dx} = 0$
TOTAL ANNUAL COST (INTEREST ON INSTALLATION + PUMPING) OF DELIVERING 1 CU. FT. PER SECOND THROUGH 1 FOOT OF PIPE AGAINST FRICTION, C = 100
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
0 10 20 30 40 50 60

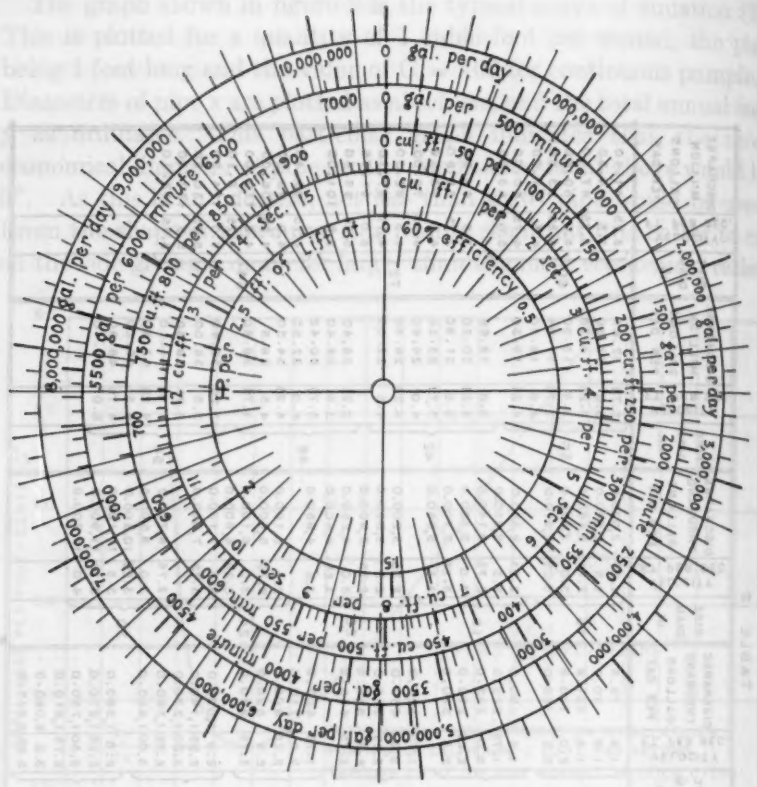
than on the side which is smaller than the most economical. This is due to the fact that a pipe which is an inch too small gives a total cost larger than a pipe which is an inch too large.

Table 2 gives the velocity in feet per second and the corresponding discharge of pipes within the range of the economical velocities covered by the friction factors C from 80 to 140 inclusive which are herewith considered.

If the material to be pumped is sewage, or water containing any considerable amount of settleable solids, the economical velocities, as herewith given, would not apply below 2 feet per second, as this

TABLE 2

| PIPE DIAM. IN. | VELOCITY FT. PER SEC. | DISCHARGE GALLONS PER DAY | PIPE DIAM. IN. | VELOCITY FT. PER SEC. | DISCHARGE THOUSAND GALLONS PER DAY | PIPE DIAM. IN. | VELOCITY FT. PER SEC. | DISCHARGE THOUSAND GALLONS PER DAY | PIPE DIAM. IN. | VELOCITY FT. PER SEC. | DISCHARGE MILLION GALLONS PER DAY |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1/8 | 0.6 | 155 | 1/4 | 1.0 | 6,700 | 4 | 1.4 | 79.5 | 16 | 2.25 | 2,020.0 | 36 | 2.75 | 12.65 | 60 | 3.25 | 41.40 |
| | 0.8 | 210 | | 1.2 | 8,050 | | 1.6 | 90.0 | | 2.5 | 2,240.0 | | 3.0 | 13.80 | | 3.5 | 44.30 |
| | 1.0 | 260 | | 1.4 | 9,350 | | 1.8 | 101.9 | | 2.75 | 2,460.0 | | 3.25 | 14.78 | | 3.75 | 47.50 |
| | 1.2 | 310 | | 1.6 | 10,700 | | 2.0 | 113.8 | | 3.0 | 2,680.0 | | 3.5 | 15.90 | | 4.0 | 50.60 |
| | | | | 1.8 | 12,050 | | 2.2 | 125.0 | | 3.25 | 2,910.0 | | 3.75 | 17.04 | | 4.25 | 53.80 |
| 1/4 | 0.8 | 375 | 1/2 | 1.3 | 11,850 | 6 | 1.5 | 190.0 | 18 | 2.25 | 2,560.0 | 42 | 3.0 | 18.66 | 72 | 3.5 | 64.20 |
| | 0.9 | 420 | | 1.4 | 12,800 | | 1.75 | 225.0 | | 2.5 | 2,850.0 | | 3.25 | 20.20 | | 3.75 | 68.80 |
| | 1.0 | 470 | | 1.6 | 14,600 | | 2.0 | 255.0 | | 2.75 | 3,140.0 | | 3.5 | 21.80 | | 4.0 | 73.50 |
| | 1.1 | 515 | | 1.8 | 16,450 | | 2.25 | 290.0 | | 3.0 | 3,420.0 | | 3.75 | 23.30 | | 4.25 | 78.00 |
| | 1.2 | 560 | | 2.0 | 18,280 | | 2.5 | 320.0 | | 3.25 | 3,700.0 | | 4.0 | 24.80 | | 4.5 | 82.50 |
| 3/8 | 0.8 | 690 | 2 | 1.3 | 18,300 | 8 | 1.75 | 395.0 | 20 | 2.25 | 3,200.0 | 48 | 3.25 | 26.40 | 84 | 3.75 | 93.40 |
| | 0.9 | 780 | | 1.4 | 19,700 | | 2.0 | 450.0 | | 2.5 | 3,550.0 | | 3.5 | 28.00 | | 4.0 | 99.50 |
| | 1.0 | 870 | | 1.6 | 22,500 | | 2.25 | 510.0 | | 2.75 | 3,900.0 | | 3.75 | 29.40 | | 4.25 | 106.00 |
| | 1.2 | 1,040 | | 1.8 | 25,300 | | 2.5 | 585.0 | | 3.0 | 4,250.0 | | 4.0 | 32.50 | | 4.5 | 112.00 |
| | 1.4 | 1,210 | | 2.0 | 28,100 | | 2.75 | 625.0 | | 3.25 | 4,600.0 | | 4.25 | 34.50 | | 4.75 | 118.10 |
| 1/2 | 0.9 | 1,240 | 2 1/2 | 1.5 | 33,000 | 10 | 1.75 | 820.0 | 24 | 2.5 | 5,100.0 | | 4.5 | 36.5 | | 5.0 | 124.50 |
| | 1.0 | 1,370 | | 1.6 | 35,200 | | 2.0 | 710.0 | | 2.75 | 5,600.0 | | 4.75 | 38.60 | | 5.25 | 130.80 |
| | 1.2 | 1,640 | | 1.8 | 39,700 | | 2.25 | 800.0 | | 3.0 | 6,100.0 | | | | | 5.5 | 137.00 |
| | 1.4 | 1,910 | | 2.0 | 44,000 | | 2.5 | 890.0 | | 3.25 | 6,600.0 | | | | | 5.75 | 143.10 |
| | 1.5 | 2,050 | | 2.2 | 48,500 | | 2.75 | 980.0 | | 3.5 | 7,100.0 | | | | | | |
| 3/4 | 0.8 | 1,930 | 3 | 1.6 | 52,260 | 12 | 2.0 | 1,080.0 | 30 | 3.75 | 7,600.0 | | | | | | |
| | 1.0 | 2,410 | | 1.8 | 59,400 | | 2.25 | 1,140.0 | | 2.75 | 8,700.0 | | | | | | |
| | 1.2 | 2,880 | | 2.0 | 66,000 | | 2.50 | 1,270.0 | | 3.0 | 9,500.0 | | | | | | |
| | 1.4 | 3,370 | | 2.2 | 72,700 | | 2.75 | 1,390.0 | | 3.25 | 10,300.0 | | | | | | |
| | 1.6 | 3,830 | | 2.4 | 79,500 | | 3.00 | 1,520.0 | | 3.5 | 11,100.0 | | | | | | |
| 1 | 1.0 | 3,660 | | 1.6 | 52,260 | 14 | 2.0 | 1,380.0 | | 3.75 | 11,900.0 | | | | | | |
| | 1.2 | 4,620 | | 1.8 | 59,400 | | 2.25 | 1,570.0 | | 4.0 | 12,700.0 | | | | | | |
| | 1.4 | 5,400 | | 2.0 | 66,000 | | 2.50 | 1,730.0 | | | | | | | | | |
| | 1.6 | 6,180 | | 2.2 | 72,700 | | 2.75 | 1,910.0 | | | | | | | | | |
| | 1.8 | 6,940 | | 2.4 | 79,500 | | 3.0 | 2,080.0 | | | | | | | | | |
| | | | | 2.6 | 86,000 | | 3.25 | 2,245.0 | | | | | | | | | |



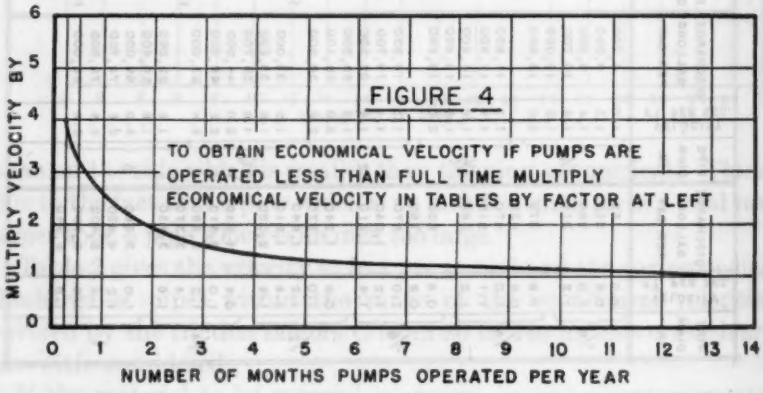



FIGURE 4
TO OBTAIN ECONOMICAL VELOCITY IF PUMPS ARE OPERATED LESS THAN FULL TIME MULTIPLY ECONOMICAL VELOCITY IN TABLES BY FACTOR AT LEFT
MULTIPLY VELOCITY BY
0 1 2 3 4 5 6
NUMBER OF MONTHS PUMPS OPERATED PER YEAR
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14

is considered the scouring velocity necessary to prevent settlement of suspended matters in the bottom of the pipe.

Use of the tables and diagrams, perhaps, can best be illustrated by examples as follows:

It is required to install a force main to discharge 50,000 gals. per day pumping 24 hours and it can be reasonably assumed that the friction factor C will not be less than 100. Referring to table 2 it is seen it will take about a 2½″ or 3″ pipe. The economical velocity of a 2½″ pipe for C 100, taken from figure 1, is seen to be 1.8 ft. per sec. But from table 2 it is seen that at this velocity it will discharge only 39,700 g.p.d. From figure 1 a 3″ pipe is seen to have an economical velocity of 1.9 ft. per sec. at C 100, and from table 2 at this velocity it will discharge about 62,000 g.p.d. Therefore a 3″ pipe should be used.

A pipe to carry 12,500,000 g.p.d. is required in use 24 hours a day under a value of C of 90. Table 2 indicates a 30″ or 36″ pipe. Economical velocity, (figure 1) for 30″ pipe, C 90 is 3 ft. per sec., and discharge at this velocity, table 2, is 9.5 m.g.d. Proceeding in the same manner with table 2 and figure 1 for the 36″ pipe it is found that it will discharge economically about 14 m.g.d., and is the size pipe required.

Next assume a pumpage of 2,125,000 g.p.d. through a line expected to maintain a value of C not lower than 125, and that the aggregate pumping time in a year is only 10 months. In other words pumps are to be operated only about 20 hours a day. From figure 4 it is seen that for 10 months the economical velocity can be increased by multiplying the 12 month value by 1.13. Again entering table 2 it is seen by inspection that the pipe will be 16″ or possibly 14″. From figure 1 economical velocity for 14″ at C 125 is 2.9 ft. per sec., and multiplying by 1.13 becomes 3.27 ft. per sec. At this velocity discharge of 14″ is a little better than 2.125 m.g.d. Therefore a 14″ pipe would be sufficient under these conditions. If, in the last example the value of C was as low as 80, it would be necessary to install a 16″ pipe for economy.

The writer desires to make acknowledgment to the standard engineering handbooks for basic data upon which this paper is based.

# THE VALUE OF THIOSULPHATE-TREATED BOTTLES FOR COUNTS ON SHIPPED SWIMMING POOL SAMPLES

By Cassandra Ritter

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With the increasing popularity of swimming as a recreation, the desirability of standard regulations as to construction, operation and laboratory control of swimming pools has become evident. Many water-works men, called upon for laboratory assistance in the control of swimming pool waters, are concerned regarding some of the special problems in this phase of bacteriology.

The revised Kansas regulations of June 28, 1935 provide the registration of all swimming pools with the State Board of Health and the local county health officer, as well as weekly bacteriological analysis in a local laboratory, or in the State Water Laboratory at Lawrence. Since local laboratories are few, samples from many of the pools are sent to the State Laboratory.

It has been demonstrated that it is possible to meet the standards of Kansas, which consider a pool satisfactory when the majority of samples contain less than 200 bacteria per cubic centimeter growing on agar at 37°C. in 24 hours, and a satisfactorily low coli content, and also the similar standards proposed in the 1935 Report of the Joint Committee on Bathing Places of the American Public Health Association and the Conference of State Sanitary Engineers (1).

Chlorine is used for the disinfection of swimming pools as it is for drinking water, but for a different reason. Quoting Mallmann and Cary (2)—"In swimming pool disinfection, the residual chlorine must destroy large numbers of bacteria as they are removed from the bodies of the bathers." These authors showed that "Samples of swimming pool water, collected during periods of use and tested immediately, showed more pollution than duplicate samples handled in the usual manner of storing and testing later," and they recommended the collection of pool samples in bottles treated with sodium thiosulphate,

* Presented before the Missouri Valley section, October, 1936.

to reduce the residual chlorine. Fletcher and Link (3) report that "A moderate excess of sodium thiosulphate when used as an anti-chlor was found to have no disinfecting effect."

As an outcome of such experimental work, the report on Bathing Places (1) provides: "All bottles of chlorinated swimming pool water shall be collected in bottles treated with sodium thiosulphate . . . to reduce the chlorine present in a treated water at the moment the sample is collected to prevent a continuance of the killing action of the chlorine on the bacteria while the sample is being transported to the laboratory."

This method was designed for analyses made soon after the collection of the sample and it has been successful for such use. Johnson (4) reports a long series of duplicate analyses but does not mention the time elapsed between collection and examination. In the Kansas water laboratory, we have the unavoidable delay from shipping, for all samples. The residual chlorine in the swimming pool water has time to complete its sterilizing effect, but dechlorinating the sample before shipment allows the bacteria to multiply greatly.

In the summer of 1936, we arranged to analyze swimming pool samples in duplicate. Bottles were prepared with a sodium thiosulphate solution as directed: one ml. of a solution of 1.5 grams of sodium thiosulphate in 100 ml. of distilled water to a sample bottle of 200 ml. capacity, added before sterilization. These sterilized treated bottles were sent with the usual sterilized, untreated bottles, to a number of pools for their weekly samples, and were returned to the laboratory in uniced cardboard containers. Usually two samples were collected from each pool, one from the shallow and one from the deep end. The time of collection and the ortho-tolidine reading for residual chlorine were requested for each set. At the laboratory, the samples were tested for total count and coli content in the usual way. The time elapsed between collection of the sample and plating was recorded.

Undetermined and uncontrollable factors having a bearing on the results are uniformity of sampling time and amount of residual chlorine present. Some pools do not chlorinate continuously and others do not have uniform distribution. Because of a lack of equipment at the pools for making the ortho-tolidine test, our reports of residual chlorine are incomplete.

During the summer, 145 analyses were made in duplicate, being 290 tests. They were from 25 different pools in 80 sets, some sets

being four samples and others two. A number of samples were collected by Dr. E. L. Treece, Special Inspector, and have the advantage of an accurate residual chlorine reading.

The 25 pools represent all types of construction. Seventeen had recirculation and eight were fill-and-draw. A single pool furnished seventeen samples; the numbers from others varied from 6 to 14; from eleven pools there were but one sample each.

Since a plate count of 200 bacteria per cubic centimeter is our standard, the results were divided on that basis. These results are shown in table 1. From a total of 145 samples collected in thiosulphate-treated bottles, 33 or 22.8 percent showed counts under 200. The duplicate samples collected in untreated bottles showed 82 or

TABLE 1

*Swimming pool samples collected in duplicate in bottles treated with sodium thiosulphate, and in untreated bottles*

| Type of pool.. | FILL AND DRAW | | RECIRCULATION | | TOTAL | |
|---|---|---|---|---|---|---|
| Number..... | 8 | | 17 | | 25 | |
| Number of samples.... | 43 | | 102 | | 145 | |
| Number and per cent..... | *under | over | under | over | under | over |
| Treated samples. | 9 (21%) | 34 (79%) | 24 (23.5%) | 78 (76.5%) | 33 (22.8%) | 112 (77.2%) |
| Untreated samples. | 22 (51%) | 21 (49%) | 60 (59%) | 42 (41%) | 82 (56.5%) | 63 (43.5%) |

* Total count under the standard maximum of 200, under; over 200, over.

56.5 percent under 200. Counts over 200 were obtained from 112 or 77.2 percent of treated samples, and only 63 or 43.5 percent of untreated ones. Thus it seems that a rather small number of dechlorinated samples reached us with a bacterial count within the standard; while more than half of those collected in untreated bottles, did. Divided on the basis of type of pool, satisfactory counts from the recirculation type of pool are slightly greater.

The residual chlorine content at the time of collection of the sample is an index as to the quality of the pool water, and directly related to the total count in the samples we receive. Ortho-tolidine readings were received for only 68 of the 145 sets of samples, the results given

in table 2. The larger amounts of chlorine had a decided sterilizing effect, shown by the large proportion of untreated samples reaching us in good condition. The 0.3 and 0.2 p.p.m. amounts were less effective. Small amounts of chlorine, 0.1 p.p.m. or less, were shown in 21 cases, not included in the table. Only 2 of these samples had counts under 200, these being untreated samples.

Another important factor is the amount of time elapsed in shipping. This varied from 12 to 93 hours, although 44 samples came in 24 hours or less and 23 others were received in 48 hours. A few samples taken from a local swimming pool showed comparable counts, all under 200, from treated and untreated bottles. A short series collected nearby in an iced container and received within four hours showed great similarity in the counts from treated and untreated

TABLE 2

*Correlation of total counts from swimming pool samples collected in thiosulphate-treated and untreated bottles, with residual chlorine present at time of collection*

| Residual chlorine..... | 0.5 p.p.m. | | 0.4 p.p.m. | | 0.3 p.p.m. | | 0.2 p.p.m. | |
|---|---|---|---|---|---|---|---|---|
| Number of samples..... | 14 | | 12 | | 7 | | 14 | |
| | *under | over | under | over | under | over | under | over |
| Treated samples..... | 2 | 12 | 2 | 10 | 2 | 5 | 5 | 9 |
| Untreated samples... | 11 | 3 | 10 | 2 | 4 | 3 | 7 | 7 |

* Total count under the standard maximum of 200, under; over 200, over.

bottles, and a direct relationship between total count and residual chlorine.

The total counts have been classified as over and under 200. Low counts are usually well under 200, as few as 3 or 4, and the majority of the high counts are several thousand. The coli content of this series of samples is quite low, only 20 of the 145 showing the presence of coli-aerogenes organisms by the partially completed test in the 10 ml. and smaller portions examined. In 7 cases, the coli index is slightly higher in the treated sample, but this series is too small to be considered more than an indication on that point.

In using these counts for practical purposes, we should consider the probable effect if all samples had been collected in sodium thiosulphate-treated bottles. Forty-nine samples, 33.8 percent, would have to be added to the 43.5 percent with counts over the standard

200. When such a count as 3000 was obtained there would be no way of knowing whether that indicated the bacterial content of the pool at that time, or multiplication in transit. The coli content is a less simple matter. The collection in treated bottles would seem to give a more accurate result; however, in this series, the presence of coli organisms in the amounts of water examined was always accompanied by a high count, which also indicates its unsanitary quality.

## CONCLUSIONS

1. In the 145 shipped swimming pool samples received during the summer of 1936 and tested in duplicate, in bottles treated with sodium thiosulphate and in untreated bottles, total counts under 200 were obtained in 22.8 percent of the treated samples and 56.5 percent of the untreated samples.

2. When swimming pool samples are subject to a delay in reaching the laboratory, due to shipping, the use of sodium thiosulphate-treated bottles leads to inaccurate results. From this study of 145 samples, it is concluded that only untreated bottles should be used for shipped samples.

3. A small series of samples from a local pool, and a series reaching the laboratory within 4 hours after collection, in an iced container, showed but little difference in the total counts of samples collected in treated and untreated bottles.

4. Swimming pool samples subjected to a delay from shipping can reach the laboratory in a condition to meet the standard of a maximum count of 200 bacteria per cubic centimeter. This was shown by 56.5 percent of the 145 samples, and by certain individual pools, which consistently met the standard throughout the season.

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# MacCONKEY'S BILE SALT LACTOSE BROTH AND STANDARD LACTOSE BROTH COMPARED AS PRESUMPTIVE MEDIA FOR WATER ANALYSIS*

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Raghavachari (1936), in a comparative study of standard lactose broth and other American presumptive test media with MacConkey's bile salt lactose broth on Indian water supplies, obtained results indicating that MacConkey's medium was superior as a selective enrichment medium in water analysis. He suggested in view of the results obtained in his study of these numerous presumptive test media that the "Standard Methods Committee" conducting the coöperative study on different presumptive test media in America, should include MacConkey's medium in their investigation.

The author (1936) prior to the appearance of Raghavachari's article had included MacConkey's bile salt lactose broth medium in a comparison of nine presumptive test media. While MacConkey's broth was not used throughout the entire study, the results obtained indicated that it was inferior to standard lactose broth and ranked fifth in the order of desirability among these nine media. In view of Raghavachari's (1936) claims this problem was further investigated employing a larger number of tests to determine if MacConkey's medium was as efficient as standard lactose broth when used as a selective enrichment medium.

## PROCEDURE

The relative efficiency of standard lactose broth, MacConkey's medium and the slightly modified MacConkey's medium as used by Raghavachari, were compared employing pure cultures of the coli-aerogenes group and also in the actual bacteriological analysis of water samples. Standard lactose broth was prepared according to the 'Standard Methods' (1933) procedure whereas MacConkey's

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medium according to the directions of the Ministry of Health, Report No. 71, (1934) and the modified MacConkey's according to Raghavachari's formula (1936). These media were sterilized at 15 pounds pressure for 15 minutes. The seventeen pure cultures of the coli-aerogenes group used in this study were isolated from water, milk and human feces. Grouped according to their biochemical characteristics nine cultures were found to belong in the genus *Escherichia*, four strains in the genus *Aerobacter* and the remaining four cultures were classified as intermediates.

The method adopted for the determination of the coli-aerogenes productivity was that of Butterfield (1933) and Hoskins (1933). The data presented have been computed on the basis of the total number of tubes showing gas, regardless of dilution. The total

TABLE 1

*Showing the Relative Efficiency of Standard Lactose Broth and the two MacConkey's Media in the Detection of Small Numbers of the Coli-Aerogenes Group*

| BROTH MEDIUM | TOTAL NO. POSITIVE TUBES | PERCENTAGE LACTOSE BROTH POSITIVES |
|---|---|---|
| Standard Lactose Broth | 350 | 100.0 |
| MacConkey's Modified Broth | 211 | 60.3 |
| MacConkey's Broth | 206 | 58.9 |

number of tubes of standard lactose broth showing gas within 48 hours is given a value of 100. The value of the other media is calculated as percentage of lactose broth positives, and shows the relative efficiency of each medium in the detection of small numbers of the coli-aerogenes group when these organisms are present in pure cultures. The results obtained upon inoculating these seventeen pure cultures of the coli-aerogenes group into the three presumptive test media are shown in table 1.

Table 1 shows that when small numbers of the coli-aerogenes group of organisms were placed in a total of 765 tubes of each medium, that growth and gas formation occurred in 350 tubes of lactose broth to only 211 and 206 in MacConkey's modified medium and the original MacConkey's broth respectively. These figures are in agreement with the earlier results of the writer except that the relative efficiency of the MacConkey's media was much less than was

found in the earlier work. The difference between MacConkey's medium as recommended in Bulletin No. 71 and Raghavachari's modification, which differs from the former medium only in that it contains 5.0 grams less of lactose and no sodium chloride, is practically negligible as shown by the above figures.

### ANALYSIS OF WATER SAMPLES EMPLOYING THESE THREE MEDIA

Ninety-four surface, shallow and deep well water samples were inoculated into standard lactose broth, MacConkey's medium and Raghavachari's modified MacConkey's medium. Five tubes of each medium were planted with 10.0 cc., 1.0 cc. and 0.1 cc. portions of water. The stronger concentration of media ingredients were employed for the 10.0 cc. portion and the single or more dilute for

TABLE 2

*The Results Obtained upon Planting Ninety-Four Water Samples into Three Presumptive Test Media Followed by Standard Confirmatory Procedures*

| | STANDARD LACTOSE | MACCONKEY'S | RAGHAVACHARI'S |
|---|---|---|---|
| Number of doubtful and positive presumptive tests within 48 hours | 70 | 64 | 70 |
| Number of confirmations | 67 | 64 | 65 |
| Percent of doubtful and positive presumptive tests confirmed | 95.7 | 92.7 | 92.8 |

the smaller amounts of water. These media were examined for gas production within twenty-four and forty-eight hours. One tube of each dilution of all three presumptive test media was carried through the partially confirmed and completed tests according to Standard Methods procedure. Table 2 shows the results of inoculating these ninety-four water samples into these three presumptive test media and the confirmation of this presumptive evidence by Standard Methods procedure. This table shows that standard lactose broth is superior to Raghavachari's modification of MacConkey's bile salt lactose medium. The percentage variation in the doubtful and positive and presumptive tests confirmed is slight but indicates that standard lactose broth in this study is the most efficient medium in the detection of members of the coli-aerogenes group in water samples.

## DISCUSSION AND CONCLUSIONS

The claim made by Raghavachari (1936) that standard lactose broth is a less efficient presumptive test medium than his modified MacConkey's medium does not appear to be valid from this study of seventeen pure cultures of the coli-aerogenes group and ninety-four water samples. Ruchhoft (1936) studied Raghavachari's modified MacConkey broth on chlorinated water samples and found it less efficient than standard lactose broth. This study of untreated water supplies has shown that MacConkey's medium is inferior to standard lactose broth. In view of the results of this study confirming the earlier work of the writer, the desirability rating of MacConkey's medium as listed in the earlier work appears correct.

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## NEW PUBLICATIONS

### Soil Corrosion Studies*

In 1922, the Bureau of Standards undertook an investigation of the effects of various types of soil on ferrous and non-ferrous metals. More than 30,000 specimens were buried in soils over the entire country and after 12 years, they were exhumed and their condition observed. The summary of the observations on ferrous metals as found in Research Paper 883 is as follows:

"1. Serious corrosion of underground pipes frequently occurs in the absence of stray currents from electrical generating stations. Electrical currents which did not originate in power houses have been observed on many pipe lines.

"2. Soil conditions have a greater effect than the type of pipe material in determining the rate of corrosion of ferrous pipes in the absence of stray electric currents.

"3. There is a relation between the average rate of corrosion of iron in soil and the soil type, but the dispersion of the data which make up the average rate is very large. This is because conditions within a soil type are not always the same. It follows that while a soil type may be designated as noncorrosive, a pipe in that soil may develop a single leak within a few years, but many leaks are not to be expected.

"4. Generally speaking, the larger the area from which the deepest pit is chosen the deeper the pit. This fact has an important bearing on the determination of the condition of a pipe line by means of local inspections. In order to make pit-depth measurements comparable, similar methods of inspecting pipes must be used.

"5. The relation of the pit depth to the duration of the exposure depends on soil conditions. In some types of soil the penetration is nearly proportional to the time of exposure, but in other soils pit depths deepen very slowly after soil conditions

*Research Papers RP 883 (May 1936) and RP 945 (November 1936) by Kirk H. Logan from Journal of Research of the National Bureau of Standards, Department of Commerce, Washington, D. C.

have become stable and corrosion products have formed on the surface of the pipe. This fact has an important bearing on the estimation of pipe life. In general, the life of a pipe should not be estimated solely from its age and the depth of the deepest pit because in most soils the rate of corrosion decreases with the time of exposure.

"6. The distribution of corrosion usually tends to become more uniform as the age of the pipe increases.

"7. Under similar soil conditions the wrought iron and steel specimens corroded at approximately the same rates during the first 12 years of exposure. In many of the soils under tests, especially those in the arid regions containing large amounts of soluble salts, the rate of corrosion of cast iron is somewhat greater than that of steel. From a practical standpoint the thickness of the material and the nature of of the corrosion products should also be taken into account in determining the material most suitable for some soil condition.

"8. The addition of copper alone to steel does not increase its resistance to the action of most soils.

"9. The rate of corrosion of pipe underground is not greatly affected by the removal of mill or foundry scale, but the available data seem to indicate a slight reduction in loss and penetration when the scale is removed.

"10. No material has been found which will withstand corrosion under all soil conditions.

"The following tentative deductions are based on a relatively few specimens exposed to 14 corrosive soils for approximately 2 years.

"1. The addition of chromium to steel reduces the rate of loss of weight but does not prevent serious localized pitting in all soils. The pitting seems to be worst in soils containing chlorides.

"2. The addition of nickel to iron-chromium alloys seems to improve their resistance to underground corrosion.

"3. An alloy containing 18 percent of chromium and 8 percent of nickel corroded less than an alloy containing more of these elements."

The observations on non-ferrous metals are summarized as follows in Research Paper 945:

"The data on the nonferrous specimens were insufficient to

justify positive conclusions as to the rates of corrosion of the materials under any specific soil condition. However, the following conclusions are suggested by the data.

"1. No metal or alloy has been found superior to all others for all soil conditions, but for each condition some suitable material is available.

"2. With but few exceptions the rates of loss of weight and penetration were less for the nonferrous specimens than for the ferrous specimens subjected to the same soil conditions.

"3. Lead in contact with some soils became badly pitted within a few years. Until the causes of this pitting are better understood, cable sheaths should be placed in direct contact only with soils known to be noncorrosive.

"4. The differences in the rates of corrosion of different varieties of lead were small but the purest lead corroded the least. Antimonial lead appeared to corrode at a slightly higher rate than the commercial lead cable sheaths.

"5. The rate of corrosion of lead gradually decreased in some soils but appeared to be maintained in certain soils such as tidal marshes.

"6. The presence of chlorides, bicarbonates, and particularly sulfates, in soils favors the formation of films or deposits which retard corrosion of lead.

"7. Copper and alloys high in copper corroded slowly in most soils. The highest rates of corrosion of copper occurred in soils containing sulfides.

"8. The rates of corrosion of all copper alloys were higher in cinders than in any type of soil tested.

"9. The corrosion of copper and high-copper alloys was more nearly uniform over the surface than was the corrosion of ferrous materials in most soils.

"10. Muntz metal dezincified in a considerable number of soils and probably should not be exposed to severe soil conditions.

"11. The rates of corrosion of copper and brass appeared to change less with time than did the corresponding rates for ferrous materials.

"12. Zinc corroded rather rapidly in a few soils. In some soils the loss of weight of zinc was nearly proportional to the duration of the exposure. In such soils effective protective films were not formed. The protection which a zinc coating affords

under such soil conditions is the result of cathodic protection at the expense of the zinc or due to the corrosion resistance of the zinc-iron alloy layers.

"13. Aluminum and some of its alloys corroded rapidly under most of the soil conditions to which they were exposed.

"14. The interconnection of dissimilar metals exposed to soil resulted in the partial protection of one of them and in more rapid corrosion of the other. Sometimes, however, the formation of corrosion products of one of the materials prevented continued corrosion."

Two types of non metallic pipe were also studied for a shorter period;—a wood fibre material coated inside and out with a bituminous substance and a portland cement (85%)—asbestos fibre (15%) material.

No general conclusions are drawn from the studies of either material since the manufacturers of each record an improvement in their product since the beginning of the study which eliminates deficiencies observed in the test specimens.

The entire study has been highly valuable and represents a thoroughly proper activity of a governmental department. It is obvious to persons acquainted with the nature of the work that neither private nor voluntary association enterprise could develop data to the extent and in the manner that they have been developed by the Bureau of Standards.

HARRY E. JORDAN.

# ABSTRACTS OF WATER WORKS LITERATURE

**Key:** JOURNAL of the American Water Works Association, 29: 1, 10, January, 1937. The figure 29 refers to the volume, 1 to the number of the issue, and 10 to the page of the JOURNAL.

## DISTRIBUTION SYSTEMS AND SERVICE PIPING

**Distribution System Problems of the New Haven Water Company.** EDWARD E. MINOR. Jour. New Eng. W. W. Assoc., 50: 3, 274–82 (1936). Suggestions are given for meeting distribution system problems, based on experiences in 600 mi. system of the New Haven Water Co. Reservoir intakes have been changed from screened pipes to concrete structures with gates and screens over openings. Screens with holes ½ in. in diam. did not stop all eels and fish until flat screens were hung on outside wall of gate house. On main feeder lines control valves are placed not over 2000 ft. apart. On a 48 in. line control gates are 42 in. and are motor driven, and are operated at least twice each month. On all other feeders gates are gone over at least once each year. System is periodically flushed. Leakage has been kept down by regular inspections. Cross connections are again more prevalent due to water from wells being used in air conditioning units and this danger must be guarded against. Used pipe is reclaimed in New Haven, and experience has shown application of final lining just before laying of pipe is worth while procedure. This is now also done on new pipe. Author recommends adequate system records be kept and describes system in use in New Haven. Also recommends study be given to available mechanical equipment that is made and which is being newly introduced to get full assistance of such devices.—*Martin E. Flentje.*

**Required Fire Flows.** P. C. CHARNOCK. Jour. New Eng. W. W. Assoc., 50: 3, 249–54 (1936). Fire flow requirements will vary from a min. of about 1000 g.p.m. in mercantile districts of small communities up to 20,000 g.p.m. in very large cities, with a low figure of 500 g.p.m. for residential areas and up to 6000 g.p.m. in tenement areas. Importance of elevated storage is pointed out as well as necessity to maintain supply in case of breakdown, and the need for adequate purification facilities to meet even unusual conditions. Figures for various requirements are given.—*Martin E. Flentje.*

**Automatic Sprinkler Systems and Their Water Supplies.** HENRY E. HALPIN. Jour. New Eng. W. W. Assoc., 50: 3, 255–66 (1936). Even though in use since 1874, public in general are ignorant of the wonderful record, value and functions of automatic sprinklers. Sprinklers discover and control incipient fires. In 69,000 fires in sprinklered properties 80 percent were extinguished or held in check by 8 sprinklers or less, 85 percent by 12 or less, 90 percent by 20 or less and 95 percent by 50 or less. Article describes and illustrates button type

sprinkler, lever and link type and bulb type. Lab. tests for approval require sprinkler (1) must not leak at 500 lb. per sq. in. water pressure; (2) soldered joint must be strong enough not to give way prematurely; (3) must give uniform distribution of water on both ceiling and floor; (4) must withstand reasonable abuse; (5) must open within 5 F. of temp. rating. Ave. discharge of sprinklers per head is 12 g.p.m. at 5 lb. per sq. in. pressure, 25 g.p.m. at 20 lb., 41 g.p.m. at 50 lb. and 55 g.p.m. at 90 lb. Under wood plank sprinklers are spaced to give a max. of 100 sq. ft. per head but never over 12 ft. apart in bays. Under reinforced concrete figures are 100 sq. ft. per head and spaced not over 10 ft. apart. Various control and alarm valves are described as well as dry-pipe systems for unheated properties. A primary and secondary water supply should be provided. At water pressure of 10 to 25 lbs. per sq. in. a sprinkler head will wet a smooth flat ceiling 4–6 in. above from 6 to 8 ft. in diam. Flowing pressure of 5 lbs. per sq. in. at head is a min. to be considered for good distribution. Sprinkler systems should be inspected once each week with periodical inspections desirable by insurance company inspector.—*Martin E. Flentje.*

**Water Department Regulation of Water Pipes Within Buildings.** STEPHEN H. TAYLOR. Jour. New Eng. W. W. Assoc., 50: 2, 129–32 (1936). The water dept. of New Bedford, Mass. has adopted a number of rules and regulations governing the installation of house piping for the purpose of providing a better supply to the consumer, preventing possibilities of dangerous back-siphonage, and for aiding the work of the water dept. This work has been placed under the full time supervision of a master plumber. With no new building going on over 700 permits were issued for alterations and replacements in 8½ mo. since June 1935. Author believes similar regulations and care should be given this problem by all communities, and that co-operation with manufacturers should be achieved to manufacture and sell only those fixtures that will not cause pollution. *Discussion:* JAMES F. MURPHY. Ibid. 133–42. This discussion by the master plumber in charge of inspections of New Bedford Water Dept. gives a number of illustrations of possible means of back-siphonage into water systems. An expt. with a hot water boiler is described that should be of interest to those troubled with excessive pressures in these boilers which at times causes damage of meters by hot water.—*Martin E. Flentje.*

**Materials Available for Water Pipes.** R. SCHEMEL. Gas- u. Wasserfach, 78: 971–4, 1935. Better asphalt coatings can be secured on cast iron pipes by preheating pipe before dipping. Steel pipe larger than 2 inches can be coated centrifugally with hot mix of petroleum asphalt with filler such as rock dust and softener such as paraffin wax. Pipes from 1 to 2 inches can be coated by rotating and using asphalt mix containing softener but no filler, but smaller pipes cannot be coated (internally) with any satisfaction, as dipping gives very uneven coatings. External coatings should be protected with heavy paper or jute wrappings. All cut pipe surfaces must be dipped in hot bitumen or otherwise protected.—*R. E. Thompson.*

**Elevated Water Tanks Cut Costs.** CHARLES BROSSMAN. The American City, 51: 8, 75, August 1936. Elevated water tanks for maintaining pressure

are a valuable adjunct to any centrifugal pumping system because:—(1) pumping need only be done at rates at which efficiency of pump is highest; (2) smaller sized pumps may be used, since peak load pumping is eliminated; leading in turn to lower electric rates, through more even distribution of power demands; and (3) pressures are maintained when pumping service is interrupted. By installing elevated tanks, Albion, Ind., cut its power bill for pumping in half and Goshen, Ind., reduced pumping costs from 8.3 to 2.2 cents per 1000 gallons.—*Arthur P. Miller.*

**Removal of Water from Mains by Compressed Air.** ROGER W. ESTY and NELSON BOARDMAN. Water Works and Sewerage, 83: 6, 193, June 1936. Compressed air, pumped into hydrant of valved-off main section, will rapidly force water out through another hydrant. Discharge hydrant is selected at lowest point of main. By this method, 6000 feet of 6- and 8-inch pipe were dewatered in 35 minutes.—*H. E. Hudson, Jr.*

**Glass Pipe Carried Water to Union College in 1840's.** RUSSEL A. HALL. Civil Eng., 6: 11, 778 (1936). Finding of several short pieces of glass tubing led to explanation of historical notes telling of glass pipe used to bring water to Union College in 1840's. Glass pipe was evidently used as supply main. For joints tubing was drawn thick at one end with a slight internal taper, and other end drawn thin and tapered to fit inside it. Plaster of Paris used for seal, and tubing encased in shell of lime or cement mortar.—*Martin E. Flentje.*

**Continuous Welding of Exposed Mains as Applied to the Gold-Fields' Water Supply.** N. FERNIE and R. J. KEATING. J. Inst. Engrs. Australia, 7: 409–20, 1935. Effects of corrosion on 30-inch locking-bar steel conduit supplying eastern goldfields of Western Australia and reasons for adopting continuous welded type of construction are discussed. Design of continuous sections, method of construction, failures and troubles experienced during experimental stage, and welding processes and costs are dealt with. Away from joints, pits developed; while, in vicinity of joints, general thinning occurred. De-aëration reduced internal corrosion. Welding methods are outlined and tests on welds tabulated. Continuous welding effected considerable economy.—*R. E. Thompson.*

**Water Mains May be Extended Outside City or Village Limits.** Health News (N. Y. State Dept. of Health), 12: 50, 197, December 16, 1935. In connection with proposed installation of sewage treatment works by city of Niagara Falls, State Department of Health suggested that less complete treatment would be necessary if Lewiston and Youngstown water supply intakes, located in Niagara River below point of sewage discharge, were eliminated and supply obtained from Niagara Falls by extension of city water mains to these communities. Attorney-General rendered opinion that proposed plan should be deemed "a city purpose" of Niagara Falls and therefore not contrary to state constitution, which prohibits a city from incurring debt except for "a city purpose."—*G. C. Houser.*

**Thawing Water Service Lines Free of Charge.** W. C. MABEE. The American City, 51: 12, 47–50, December 1936. Because of extreme cold experienced

in Indianapolis, Indiana, during early part of 1936, an unprecedented number of water service lines were frozen. To cope with these conditions, fourteen gasoline-motor-driven electric arc-welding machines were used. One terminal from generator was attached to frozen line after meter had been disconnected, and the other to nearest hydrant, curb stop key, or service pipe in an adjoining house. In this manner, as many as twenty-five services per day were thawed by one crew of operators with one machine.—*Arthur P. Miller.*

**Pipe on Long Suspension Bridge Presents Novel Design Problem.** Eng. News-Rec., 116: 525, April 9, 1936. Water for 1939 San Francisco exposition will be delivered through 10-inch main carried on that part of San Francisco-Oakland Bay bridge spanning West Bay channel. There are two 2,310-foot spans and other shorter spans, totaling 10,300, between anchorages. Bridge was designed for expansion-contraction range up to 42 inches and vertical movement up to 25 feet. Although these extremes will not occur frequently, it is desirable that pipe line shall meet these requirements.—*R. E. Thompson.*

**Elevated Tanks Strengthened after Earthquake Hazard Survey.** H. M. ENGLE. Eng. News-Rec., 116: 807–9, June 4, 1936. As result of effect of 1933 earthquake in Long Beach on roof and other elevated tanks used for fire protection of buildings, Board of Fire Underwriters of the Pacific undertook state-wide survey of elevated tanks, including structural analysis of 650, which was completed about end of 1934. Some of essential findings are given, together with remedial measures suggested to owners. Deficiency in resistance to lateral forces was found in 95 percent of all structures examined. More than 60 percent of all major tank structures in more hazardous zones have now been repaired, or removed, and it is probable that more than 85 percent of all structures examined will be so treated.—*R. E. Thompson.*

**Leakage Surveys.** J. B. EDDY. Eng. News-Rec., 116: 767–9, May 28, 1936. Since 1932, regular yearly appropriations have been made in Chicago for leakage surveys. From January, 1932, to April, 1936, 72.47 m.g.d. loss was eliminated in 796 miles of pipe tested, average of about 17 m.g.d. each year or 91,000 gallons per day per mile of pipe. Total of 4,571 leaks, with average flow of 14,300 gallons per day, were corrected. Number of men employed on this work averages 250. In addition, periodical plumbing fixture examinations are made by plumbing inspectors. Methods employed are described. Aquaphone and rod and a dye method developed by department engineers have been found most useful. Electric leak locator is too sensitive to noise from other sources. Dye method, which consists of injecting fuchsin, harmless red-colored compound, into main and observing time of flow between different points, is dealt with in some detail.—*R. E. Thompson.*

**Water Waste Survey in Lachine, Quebec.** ROBERT DORION. Eng. Contract Record, 50: 495–8, June 3, 1936. Details given of waste survey of distribution system (33.2 miles of mains 4 to 20 inches in diameter), together with results obtained. Treatment consists of rapid sand filtration (eight 1-m.g.d. units) and chlorination. Following severe winter of 1933–4, consumption increased

to 7 m.g.d. Survey disclosed 95 underground leaks with estimated loss of 1,377,000 gallons per day and 685 leaking plumbing fixtures with estimated waste of 746,000 gallons per day. On basis of cost of production of 2¢ per 1000 gallons, leakage found represents annual preventable loss of $15,497. Repairs to date have reduced waste by 1 m.g.d., estimatedly saving $6,323.78 per year. Employees have been trained in use of waste detection instruments, gages have been permanently installed, and 1 of 5 districts will be surveyed annually.—*R. E. Thompson.*

**Elevated Tanks in Earthquakes.** H. A. SWEET. Eng. News-Rec., 117: 94–5, July 16, 1936. Extended discussion of article, "Elevated Tanks Strengthened after Earthquake Hazard Survey," by H. M. ENGLE (Eng. News-Rec., June 4, 1936, p. 807). Writer believes that such a program of repairs, involving expenditure of several hundred thousand dollars, should have sounder basis than now appears. Research work to date indicates advisability of repairing towers to resist dynamic rather than static stresses. Use of data now available regarding dynamic stresses and spending of more effort and money to increase this knowledge and less in strengthening elevated tank towers on static basis are urged.—*R. E. Thompson.*

## WATER QUALITY

**Water Supplies Need Constant Vigilance.** Weekly Health Message (Iowa State Dept. of Health and U. S. Public Health Service), No. 486, February 3, 1936. Polk County, Iowa, recently experienced water-borne epidemic of typhoid fever, 2 fatalities occurring among 38 cases. Investigation revealed that combination of sewer connection to well pit, clogged sewer, and cracks in well cover permitted sewage to seep directly into well. Prompt chlorination of water was factor in preventing epidemic from becoming more widespread.—*G. C. Houser.*

**The Efficiency of Rapid Sand Filters in Removing the Cysts of the Amoebic Dysentery Organisms from Water.** JOHN R. BAYLIS, OSCAR GULLANS and BERTHA KAPLAN SPECTOR. Public Health Reports, 51: 46, 1567–1573, November 13, 1936. Experiments were made on the removal of *Endamoeba histolytica* cysts from water by rapid sand filtration. Experimental filter, with 10 square feet of surface area, had 24-inch layer of sand of effective size of 0.5 mm. Cysts were added to the coagulated water going to the filter at a constant rate from 5-gallon portions of a cyst suspension containing from 362 to 2,370 individuals per gallon. Only 4 cysts were found in 114 gallons of filtered water tested, percentage reduction being in excess of 99.99. Duration of experiments ranged from 3.7 to 13.7 hours. From 9 to 74 liters of filtered water were collected at equal intervals during experiment and run through a Foerst centrifuge for concentrating the cysts. Prechlorinating the water did not affect filter efficiency. Conclusion reached was that rapid sand filters as now used effectively remove *E. histolytica* cysts from water.—*R. E. Noble.*

**Typhoid Fever Outbreak Traced to Polluted Spring Water.** Public Health Reports, 51: 42, 1445, October 16, 1936. Between July 1 and September 7,

1936, about 60 cases of typhoid fever were reported from Bergen County, N. J., 40 of which were traced to a public spring in Englewood (actually a stone reservoir fed by a pipe and having an overflow spout). Water had evidently been polluted by human feces deposited in nearby bushy area. Rodent holes permitted surface rain water to wash pollution down to an open-jointed tile pipe conveying water from hidden spring to the reservoir. Latter was taken out of use, thus eliminating source of outbreak.—*R. E. Noble.*

**General Water Hygiene and Water-Borne Disease.** W. VON GONZENBACH. Schweiz. Verein von Gas- und Wasserfachmännern, Monatsbulletin, **16**: 8, 200, August 1936. Significance of water supply for public health is discussed. Attention is called to water-borne gastro-intestinal epidemics caused by atypical *B. coli*, such as summer diarrhea of visitors in high mountain regions after drinking from springs polluted by pasturing cattle, outbreaks of intestinal complaints among troops quartered on the frontiers due to unsafe drinking water consumed with impunity by local inhabitants, and seasonal incidence of similar epidemics during warm months. In diarrheic stools of water-borne summer diseases, slime-forming strains of atypical *B. coli* with limited ability to ferment sugars are abundantly or exclusively present. Contrary to usual acceptance of *B. coli commune humanum* as sole criterion of pathogenicity, it is believed that water containing members of the colon group as a whole is unsuitable for human use. As to pool water, infections other than conjunctivitis are unknown; bacteria in small amounts of pool water imbibed by bathers are believed to be destroyed by gastric acid; therefore, for pool water, less exacting standards for purity are justified.—*Manz.*

**Bottled Water.** T. S. KENNEDY. Health Notes (Florida State Bd. of Health), **28**: 2, 20, February, 1936. Use of bottled water for drinking in Florida is quite extensive, principally because of tastes and odors resulting from high mineral content in many municipal supplies. Rule passed by State Board of Health governs manufacture, importation, and bottling of water for sale in the State. It requires that all bottlers of water shall secure a permit from the Board before offering their product for sale and provides for revocation of such permits where deemed advisable. Permits cover bacteriological quality only: no responsibility is accepted for therapeutic claims made by bottlers.—*G. C. Houser.*

**An Epidemic of Bacillary Dysentery in Matane, Quebec.** A. R. FOLEY. Can. Pub. Health J., **27**: 113–7, 1936. Details are given of epidemic involving one-third of population (5,134) and resulting in 39 deaths, which is believed to have been water-borne. Morbidity rate was very high, particularly among infants. Causative organism was *B. dysenteriae Flexner*. One of men employed on alterations to intake system of municipal water supply was found to be chronic carrier. Epidemic subsided following disinfection of water supply system.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

**Intestinal Epidemic Origin under Study at Milwaukee.** Eng. News-Rec., **116**: 611, April 23, 1936. No conclusion has been reached regarding origin of

gastro-intestinal epidemic that affected more than 120,000 people in Milwaukee during latter part of March and early part of April. One theory suggests recent dust storms. Water supply has also been closely scrutinized and order issued to boil all water used. During past few weeks strong winds moved sewage-polluted water into vicinity of intakes, causing great variations in quality of raw water, which, in addition to large numbers of *B. coli*, also contained abnormal numbers of chlorine-resistant organisms of spore-forming type, such as *B. welchii*. Such conditions have frequently occurred in past, however, and were not considered unusual.—*R. E. Thompson.*

**The Decline of Typhoid.** M. N. BAKER. Eng. News-Rec. 117: 397–9, 1936. The annual typhoid death rate data of the American Medical Association (J. Am. Med. Assocn., June 10, 1936) for cities with population in excess of 100,000 are reclassified and discussed. It is believed that in cities where relatively high rates have prevailed the cause is other than the public water supply; *e.g.*, use of private wells and general sanitary conditions. This conclusion is supported by the number of cities with no typhoid deaths in 1935 where chlorination is the only treatment, while nearly all the cities with high rates employ filtration also.—*R. E. Thompson.*

**Report of Typhoid Epidemic in Rural County in Kentucky.** F. W. CAUDILL and PAULINE MYERS. Bulletin of the Department of Health (Kentucky), 8: 7, 151, February, 1936. Interesting phase of recent typhoid epidemic in rural county in Kentucky was sudden flare-up into epidemic proportions of disease which had been for years abnormally prevalent in community. County's normal typhoid expectancy was about 27 cases per year. During period of 12 weeks in 1935, 186 cases of typhoid fever developed, with 16 deaths, in population of 6600. During early part of June, unusually heavy rains caused waters of two streams to rise, with flood conditions prevailing for about 10 hours on June 22. Waters, loaded with mud, débris, and the contents of open, surface privies, overflowed many wells. Eventually draining away, they left water-soaked and sewage-contaminated soil. During early part of July, typhoid cases began to develop. Drop in case incidence was noted soon after wholesale chlorination of water supplies during first week in September.—*G. C. Houser.*

**Belmont County Flood Presents Serious Aspects.** Ohio Health News, 11: 15, 1, August 1, 1935. Flood, which followed torrential rain on upper portion of Wheeling Creek, Belmont County, O., July 4, 1935, wrought critical health situation. All water supplies, derived from 840 shallow dug wells, were contaminated by flood waters; not only from natural pollution, but also because, scattered over the narrow valley terrain, were pit-contents of hundreds of insanitary privies and four known cases of typhoid fever. After flood, numerous cases of diarrhea and other intestinal ailments were found. Water from approved public supply was hauled into valley, and wells were sterilized by chlorination.—*G. C. Houser.*

**Twenty Years of Sanitary Engineering.** C. G. GILLESPIE. Weekly Bulletin, California Department of Public Health, 15: 1, 1, February 1, 1936. In 1915,

California legislature provided a Bureau of Sanitary Engineering in State Board of Public Health. Through attacks by many agencies, on many fronts, greatest of which is water disinfection and better laboratory control, typhoid fever epidemics, as derived from water, have almost disappeared, the last water-borne typhoid outbreak in California having occurred in 1924. Prior to that time, scarcely a year failed to produce one or more water-borne typhoid outbreaks. First major task in 1915 was to promote safety of water supplies, state-wide bacterial control of which was at once established. Almost 150 places have provided some form of water treatment, most common of which is disinfection: there are now 36 filtration plants.—*G. C. Houser.*

**Flood Waters Endanger Health.** Weekly Health Message (Iowa State Dept. of Health and U. S. Public Health Service), No. 491, March 9, 1936. Water from well which has been flooded should not be used unboiled until after well has been cleansed and disinfected. After flood waters have receded, well should be pumped until all turbid water has been removed. For disinfecting water in an ordinary well, use ¼ to ½ pound of commercial chlorinated lime. In large-diameter wells, 1 pound may be used. Chlorine solution should remain in well at least 24 hrs. Well should then be pumped until chlorine taste no longer persists. Analysis may be made in about a week, and if contamination still persists, dosage with chloride of lime should be repeated.—*G. C. Houser.*

**Public Water Supply Emergencies.** Monthly Bulletin, Indiana Division of Public Health, 39: 4, 59, April 1936. February 1936 in Indiana was marked by much trouble from frozen service lines, tastes and odors in water supplies, clogging of intakes with ice, high turbidity and questionable bacterial quality of finished water, and raging floods. Emergency conditions prevailed in 14 water plants, serving combined population of 110,000. Minor intestinal disturbances, reaching epidemic proportions, were reported in 2 municipalities, but there was no evidence of typhoid outbreaks.—*G. C. Houser.*

## STREAM POLLUTION

**Michigan Moves Against Stream Pollution.** Eng. News-Rec., 116: 859, June 11, 1936. Michigan Stream Pollution Control Commission on May 27 issued orders to abate pollution of Saginaw watershed. Oil operators were directed to take immediate steps to return brine to formations underground. Large chemical company was ordered to provide permanent means of waste disposal satisfactory to commission. One oil company is making experiments in which brine is pumped into glacial drift about 240 feet below ground level.—*R. E. Thompson.*

**Proposed Potomac River Conservancy District.** Maryland Department of Health, Press Bulletin 601, February 3, 1936. Preliminary steps toward cooperative action relative to establishment of proposed Potomac River Conservancy District were discussed by representatives of State Departments of Health of West Virginia, Virginia, Pennsylvania, and Maryland at conference

held January 27 in Baltimore. Selection of Potomac River basin as demonstration unit for water pollution control was recommended to National Resources Committee by Advisory Committee on Water Pollution. Special consideration was given to extent of pollution of public water supplies by sewage and industrial waste, as well as to effect of such pollution upon public health and upon destruction of fish and water fowl.—*G. C. Houser.*

**W. P. A. Projects in Kentucky.** F. C. DUGAN. Bulletin of the Department of Health (Kentucky), **8**: 4, 113, November 1935. State Dept. of Health is coöperating with W. P. A. in carrying out three projects designed to protect and promote public health, comprising sanitation, mine sealing, and malaria control. Mine-sealing project, embracing 34 counties in coal-mining sections of State, will involve expenditure by W. P. A. of about $400,000. Under program mapped out, it is expected to seal approximately 13,000 abandoned coal mines in Kentucky, which are now discharging acid mine water into streams.—*G. C. Houser.*

**Pollution of Delaware River. An Interstate Problem.** Health News (N. Y. State Dept. of Health), **13**: 15, 59, April 13, 1936. Water pollution abatement in Delaware River is of vital interest to State of New York, since this river, besides being probable future source of supply for metropolitan centers in New Jersey and Pennsylvania, is a proposed source of water supply for Greater New York. The Delaware between its source and Port Jervis is relatively clean and, after proper purification, suitable as source of water supply. It would seem wise for states involved to coöperate in abatement and control of pollution. Unless federal aid in local projects is forthcoming in form of loans or grants, there will be little progress in elimination of stream pollution in U. S. for some time.—*G. C. Houser.*

**A Reservoir Silting Case.** Eng. News-Rec., **116**: 695, May 14, 1936. Neubert Springs Dam, 15-foot earth embankment built in 1921 near Knoxville, Tennessee, when washed out by flood on March 24 was found to have accumulated silt to within 2 feet of pond level, or 85 percent full. Deposit totaled about 25,000 cubic yards, or 1 acre-foot per year. Drainage area, 1100 acres, consists of 40 percent cultivated fields and 60 percent sparse second-growth timber. Remains of older dam several hundred feet upstream caught large part of gravel and heavy material, deposit described probably representing only about one-half of material brought down by stream.—*R. E. Thompson.*

**Public Waters Pollution Control in New Hampshire.** CHARLES D. HOWARD. Jour. New Eng. W. W. Assoc. **50**: 3, 334–39, (1936). Article gives details of public health laws dealing with pollution of water supplies in N. H. and shows how lakes and streams are protected under these statutes. Lists needs to make enforcement more effective.—*Martin E. Flentje.*

**Treatment and Recovery of Sulfite Waste.** F. MERRYFIELD. Civil Eng. **6**: 6, 374–76, (1936). Article is a survey of ways to alleviate stream pollution by sulfite wastes of paper mills, a problem that looms large in the industry and among sanitary engineers. The best opportunities for disposal of these wastes

seem to be utilization as cattle fodder, fertilizer and fuel. Ponding and aeration appear to have value but large scale tests are necessary before any generalization can be made as to efficiency of these methods in reducing oxygen demand.—*Martin E. Flentje.*

**Stream Cleansing in Oregon. Past and Present Efforts to Reduce Pollution.** R. E. Koon. Civil Eng. 6: 10, 678–80, (1936). 71 percent of urban population of Oregon reside in Willamette River valley, an area less than 12 percent of total area of state. No city water supply in state is now taken from a stream now grossly polluted or ever likely to be. After this year only one city will be taking water from a stream even moderately contaminated, which source is to be abandoned in the near future. Stream purification in state is desirable because of large population of nature lovers. Present laws and desirable amendments are discussed with other aspects of the problem. **Present Status of Municipal Sewage Treatment.** Carl E. Green. Ibid., 680–82. **Industrial Wastes in the Willamette Valley.** Fred Merryfield. Ibid., 682–84. Willamette River is most polluted river in Ore. and is location of major portion of industries, such as pulp and paper mills, lumber mills, creameries, woollen mills, tanneries, flax retting plants, soft fruits and vegetables. River primarily rain fed, reaches min. of 4000 cu. ft. per sec. at Salem in Sept. and Oct. Above Salem river is practically saturated with oxygen, but the 68 mi. below Salem to Portland becomes progressively worse. River is little used for water supply but is valuable as recreational area and as valuable spawning ground for salmon and game fish. Table is given showing B.O.D., solids and pH of various plant effluents with discussion of these wastes. Author believes pilot plants should be operated for further study of waste treatments.—*Martin E. Flentje.*

## WATER TREATMENT—GENERAL

**The New Water Treatment Plant of the Salem and Beverly Water Supply Board.** Frank A. Marston and John W. Raymond, Jr. Jour. New Eng. W. W. Assoc. 50: 2, III–28, (1936). An account is given of the ERA project by which an 8 m.g.d. treatment plant was constructed at a total cost of $654,149. Plant supplies Salem and Beverly, Mass., total population 69,343. Water is taken from Wenham Lake, Longham Res. and Ipswich River, during part of year. Water from these sources often high in color, with objectionable taste and odor, and corrosive to the extent that red water trouble is experienced. Plant consists of intake with screens, low lift pumping station, two 18 x 18 x 17.5 ft. water depth concrete mixing basins containing mechanical wooden agitators, 4 covered concrete coag. basins each 95 x 33 x 15 ft. avg. water depth, 8-1 m.g.d. rapid sand filters with perforated C. I. underdrains, 18 in. gravel and 30 in. sand (specified not less than 0.35 nor more than 0.45 mm. effective size), 2 wash water pumps designed to give wash water rates of 36 in. rise per min. or less. Clear well consists of a covered concrete basin with groined-arch floor and roof, 217 ft. square and 13¼ ft. of water depth, capacity 4 m.g. Chemicals are fed by dry feed machines and provision is made for chlorine and anhydrous ammonia feed. A good record of the construction of

this plant with unit labor costs, rates of pay and complete costs is given.—*Martin E. Flentje.*

**Filtration Plant of the Mystic Valley Water Company.** WALTER H. CHASE and T. B. RAY. Jour. New Eng. W. W. Assoc. 50: 3, 325–30, (1936). Water is obtained from Palmer and Dean ponds and has color from 80 to 100 and objectionable taste and odor. Velvet industries in community required treated water. 2 m.g.d. treatment plant was built in 1935, consisting of aeration, chemical treatment, sedimentation, filtration, pH correction and chlorination. Soda ash, alum and activated carbon used; for best results pH of 5.8 to 5.9 maintained in coagulated water. Color reduced by settling from 100 to 30 and by filtration to 5. pH raised finally to 7.—*Martin E. Flentje.*

**A Model of a Rapid Sand Filter Plant.** RANALD V. GILES. Civil Eng., 6: 11, 779, (1936). Description is given of a working model of a filter plant built in school shop at cost for materials not over $35.00. Model consists of baffled mixing basin, settling basin equipped with movable baffles and glass filter unit with wash water troughs and perforated underdrains. Model is used for instruction and student lab. tests.—*Martin E. Flentje.*

**A New Method of Treating Very Soft Surface Waters.** L. W. HAASE. Gesundh.-Ing. 59: 41–2, 1936. Calcium carbonate hardness amounting to at least 2 German degrees is necessary for deposition of protective deposit in mains, etc. As soft water usually requires clarification by alum treatment, it is recommended that sodium carbonate be used as alkaline agent with alum rather than lime; thus carbonate content of water is increased. Lime can be subsequently added if necessary to bring calcium content up to desired level. Use of lime as alkaline agent with alum increases only non-carbonate hardness, which is not effective in forming protective deposit. Furthermore, coagulation is better when sodium carbonate is used.—*R. E. Thompson.*

**Handling Peaks at Sacramento.** Eng. News-Rec., 116: 778, May 28, 1936. Average consumption in Sacramento is 23 m.g.d., with peak of 37 m.g.d. Filter plant and pumping station are located in extreme northwest corner of city. To increase efficiency of operation, improvements are in progress, including 25-m.g.d. low-lift pump, 10-m.g.d. high-lift pump, 8-m.g. filtered water reservoir, two 3-m.g. elevated tanks, 13,000 feet of 24-inch welded steel pipe, and other connecting mains. Reservoir will be 275 x 315 feet, with vertical walls 19 feet high. Floor and walls will be of reinforced concrete and interior columns will support wooden framework for roof of creosoted Douglas fir covered with asbestos felt. Tanks will be 146 feet in diameter and 105 feet in total height. Water depth will be 25 feet and floors will be 75 feet above ground. Superstructure will consist of tank with sides of steel plate surrounded by reinforced concrete wall. Roofs of redwood covered with asbestos felt will be carried by concrete columns within tanks. Contract price for 2 tanks was $315,347.—*R. E. Thompson.*

**Long-Delayed Filtration Plant Completed at Fort Scott, Kansas.** FREEMAN D. MARTIN. Eng. News-Rec., 117: 403, 1936. Brief description of new

4-m.g.d. plant for treating Marmaton River water, which is impounded in reservoirs holding 233 million gallons. Plant includes air-agitated mixing basins; chemical feed machines for applying chlorine, ammonia, activated carbon, alum, and lime; settling basins; and four 1-m.g.d. rapid sand filters. Detention periods provided are as follows:—mixing, 40 minutes; coagulation and settling, 14 hours; secondary mixing, 40 minutes; final settling, 9 hours. Filters contain 18 inches of gravel and 22 of sand. Automatic electrically-controlled booster station and 200,000-gallon elevated tank were also constructed for improving pressure distribution.—*R. E. Thompson.*

**Aëration and Filtration for Altamont, Illinois.** Anon. The American City, 51: 12, 61–62, December 1936. A new PWA-financed water works for this city consists of 400-foot-long dam impounding 60 million gallons of water, two pumping plants and water treatment plant. Latter comprises: two pan aërators, mixing flume where alum, lime, and activated carbon are added, sedimentation basin, two rapid sand filters, clear well, and chlorinating apparatus. All services in system are metered. Because of improved quality of the water, consumption increased to a point that warranted a decrease in water rates soon after new system was in operation.—*Arthur P. Miller.*

**Layout and Operation of Hamilton's Water Filtration Works.** W. L. McFaul. Eng. Contract Record, 50: 252–7, 1936. Detailed description of rapid sand filtration plant completed in 1933 at cost of $1,220,000, and including intake in Lake Ontario, low-lift pumping station, flash-mixers consisting of impellers on vertical shaft, mixing chambers with flocculators operating at 1 to 3 r.p.m. and allowing 30 minutes detention, settling basins allowing 3 hours detention, twelve 3.33-m.g.d. filters, 2-m.g. filtered water reservoir, and chlorination equipment. Underdrainage system is of perforated pipe type, designed for maximum wash rate of 42 inches vertical rise per minute, and filter beds are made up of 20 inches of gravel in 5 layers (2.5-inch to 10-mesh) and 30 inches of sand with effective size of 0.5 mm. and uniformity coeff. of 1.6. Rate of wash is adjusted to give sand expansion of 50 percent. Filter runs are limited to 48 hrs. or loss of head of 8 feet. Algae frequently reduce runs to 30, or even 20, hours. As raw water turbidity is low, alum is used only intermittently. Water is treated with ammonia and chlorine (1:4) prior to filtration, the residual chlorine persisting is maintained through the plant most of the time and rendering post-chlorination unnecessary.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

## MICROÖRGANISMS

**Control of Microörganisms in the Albany Water Supply.** Eng. News-Rec., 116: 487, April 2, 1936. Microörganisms have been chief source of trouble in Albany, N. Y., since introduction of gravity supply in 1932. Changing point of draw-off from 13-billion gallon Alcova Reservoir by using one or another of 4 outlet gates, some 14.5 feet apart vertically, provides some degree of control; but at times manganese content of bottom water prevents use of lower outlets. Experiments have indicated that it may be desirable to change coagulant from alum to iron salt. It was found that 1.2 grains per gallon

ferrous sulfate and 0.7 grain calcium hydroxide will reduce manganese more than 60 per cent and give effluent with color of 12 p.p.m. If algae are kept below 200 standard units per cc., water may be effectively coagulated and 28- to 65-hour filter runs obtained. Experience has indicated that most effective method of applying copper sulfate is periodically in reservoir and continuously as water enters 10-mile conduit leading from reservoir to filters. Application at conduit inlet controls chlorophyceae and cyanophyceae, but has little effect on diatoms, particularly *Melosira*, which, dead or alive, shorten runs. *Melosira* are, at best, reduced only 50 percent during 5-hour flow period through conduit. Control of diatoms, therefore, must be effected by treatment of reservoir in early spring and in late fall just prior to ice forming. In May, 1935, with application of 0.42 p.p.m. copper sulfate at conduit inlet, *Melosira* reduced runs to 6 hours. Next day the billion-gallon prism of water nearest dam was treated with 3600 pounds copper sulfate (0.5 p.p.m.) and within week runs were normal. Reservoir has been plotted and profiled in 4 zones and volume of each per foot depth computed. Chemical is applied in crystal form from speed boat. In December, 1934, *Synura* suddenly gave rise to cucumber-brassy taste. Customers' complaints began, week before organism was detected at plant. Organism is derived from swamp water, grows only in cold, and taste is caused by essential oil liberated on breaking up of cells, which are very delicate. Taste is that of ripe cucumber, which is followed by dry brassy taste in throat. On December 22, *Synura* numbered 110 standard units per cc. Application of copper sulfate at conduit inlet reduced odor 50 percent. Activated carbon was applied at aërator effluent as soon as trouble began, but was not wholly efficacious. By February 13, using 0.25 grain per gallon of carbon applied at coagulation basin outlet, trouble was completely controlled. On January 29, *Synura* was found at all depths in reservoir. In February, 1935, *Oscillaria* in 75-million gallon unit of Loudonville distribution reservoir caused complaints of musty odor. Heavy mat of *Oscillaria* and soot was found on side slopes and bottom of basin, 20 feet below water surface. Neither drawing off part of water nor heavy chlorination proved successful in combating nuisance, but copper sulfate shoveled from boats on to slopes was effective. Year's trial of this method is being made on all 3 basins. Data given were derived from report of GEORGE E. WILLCOMB, supervising chemist, for year ending September 30, 1935.—*R. E. Thompson.*

**Crenothrix Chokes Conduits.** G. E. ARNOLD. Eng. News-Rec., **116**: 774–5, 1936. About 3 weeks after Hetch Hetchy water was turned into 36-inch Alameda line crossing San Francisco Bay, carrying capacity of line started to drop off at rate of 0.2 million gallons per day and within 3 weeks had lost 16 percent. Inspection disclosed slimy gelatinous growth, light brown in color, covering entire inner surface of pipe to depth of $\frac{1}{8}$ to $\frac{1}{4}$ inch. Microscopic examination showed presence of *Schizomycetes crenothrix*. Further inspection revealed that other lines were also affected and that source of trouble was seepage into Coast Range tunnels, containing almost pure cultures of the organism. Slime growth had following percentage composition: volatile matter, 48; calcium, 26; magnesium, 3.2; iron, 1.4; aluminum, 3.9; $SO_4$, 4.9; $SiO_2$, 12.5. Water left standing in conduit developed strong objectionable

medicinal, or woody, taste. Dissolved oxygen content of water decreases 50 percent during passage through tunnels, which are 29 miles in length. Neither copper sulfate (2 pounds per million gallons), nor lime (sufficient to increase pH to 10) had any beneficial effect. Chlorine was effective as far as residual chlorine could be maintained. Treatment with chlorine and ammonia (6 and 1.5 pounds per m.g., respectively) has completely eliminated growths in Bay crossing, improvement being observed within 1 week of starting treatment. Plans are being made for treating water in tunnels, where growth remains undiminished.—*R. E. Thompson.*

## IRON AND MANGANESE REMOVAL

**Zeolites Remove Manganese.** D. E. Davis and J. T. Campbell. Eng. News-Rec., 116: 781–3, 1936. Water supply of Ambridge, Pa., derived from 29 wells tapping water-bearing sands underlying Ohio River, has an average manganese content of 5 p.p.m. and average hardness of 258 p.p.m. Purification works consist of split-treatment plant in which 75 percent of water is softened and de-manganized in three 0.5-m.g. greensand base exchange units and then mixed with remainder after latter has been treated with lime and filtered. Product is Mn-free and has hardness of 82 p.p.m. Greensand has softening capacity of 3200 grains calcium carbonate per cubic foot during "zero hardness" period and of an additional 200 grains during the "twilight" period while effluent is increasing from zero to full hardness. In the "twilight" zone, effluent is still free from manganese when hardness reaches 30 p.p.m., but contains 1.4 p.p.m. when hardness increases to 70 p.p.m. Units are backwashed at rate of 12 gallons per square foot per minute to remove adhering manganese, and then regenerated with brine. The preliminary wash increases softening capacity. The zeolite grains are free from iron or manganese deposits after nearly 2 years' operation, nor has decrease in exchange capacity been noted. Lime treatment plant consists of 30-minute mixing chambers, coke trays, submerged coke bed, settling basin with 2.7 hours retention, and 0.5-m.g. rapid sand filter. Sufficient lime is added to maintain pH value at 9.5. Prior experiments with pyrolusite had shown that approx. 2.5 p.p.m. manganese could be removed by catalytic action, but that pH had to raise to remove the remainder. This observation has been confirmed on plant scale by cutting out lime treatment for short periods. Under prevailing conditions, cost of removing hardness is identical for lime and for zeolite. Cost of plant was $124,690. Treatment has increased total cost of operating by about 2.5 cents per 1000 gallons.—*R. E. Thompson.*

**Iron and Manganese Removal at Lincoln.** Eng. News-Rec., 116: 773, May 28, 1936. Plant consisting of aërator, chlorination equipment, upward-flow gravel contact beds, 2-hour sedimentation basins, and sand filters was placed in operation in December, 1935. Iron content of well water varied from 0.4 to 0.8, and manganese content from 0.5 to 1.0 p.p.m. Effluent contains less than 0.1 p.p.m. iron and approximately 0.27 p.p.m. manganese. Manganese removal is constantly improving with increasing manganese coating on sand grains in final filters and it is expected that, after 1 year's operation, residual

manganese will not exceed 0.1 p.p.m. Chlorine dosage is 1.5 p.p.m.—*R. E. Thompson.*

**Manganese Ore for Iron and Manganese Removal.** W. G. KIRCHOFFER. The American City, 51: 10, 60, October 1936. Satisfactory removal of iron and manganese has been attained by filtering through beds of pyrolusite (manganese dioxide rock), or through filters in which higher oxides of manganese have been precipitated. It is believed that in either case the removed manganese acts as a catalyst and is unchanged as a result of the reactions. The complementary use of aëration, together with lime and alum, makes the removal more complete.—*Arthur P. Miller.*

**New Procedure for Elimination of Colored Humic Matter from Water for Drinking and Industrial Use.** F. SARTORIUS. Gesundheits-Ingenieur, 59: 42, 614, October 1936. Colored water is first aërated, then treated with readily oxidizable cast iron filings for artificial ferrization and filtered through magno-material (burnt dolomite), iron being precipitated and humic matter absorbed; by two further stages of magno, with little iron filings and pure magno a thorough purification is reached. Clear oxygen-free water with low permissible iron content is obtained. Data of experiments and suggestions for application of procedure on plant scale are given.—*Manz.*

## BOILER FEED WATER

**Cause and Prevention of Turbine-Blade Deposits.** FREDERICK G. STRAUB. Trans. Am. Soc. Mech. Engrs., 57: 447–54, 1935; Combustion, 7: 6, 23–7, 1935. Laboratory and plant-scale experiments show that turbine blade deposits result from contamination of steam by boiler water. Laboratory experiments indicate that sodium hydroxide is responsible for scale adhesion; when it is converted to sodium carbonate, scale does not adhere. Presence of sufficient amounts of neutral salts, such as sodium sulfate, also prevents scale adhesion. Deposition of water-soluble salts carried into turbine by mechanical entrainment is thus major problem. Following conclusions were drawn from study:—(1) amount of deposit is not proportional to carry-over from boiler, but depends upon composition of boiler water; (2) salts which form dry powder in superheated steam will not adhere to turbine blades; application of such salts (sulfates, chlorides, and carbonates) to boiler feed water will reduce deposits by coating particles of sodium hydroxide and allowing them to be carried on through turbine; and (3) concentrations of 4 or 5 times that of sodium hydroxide are required for effective scale prevention, if sodium sulfate is used as added salt. Cost of treatment is less than $500 per year, based on experiments in large central power plant.—*R. E. Thompson.*

**The Purification of Feed Water.** HOYER. Monatschr. Textil-Ind., 51: 17–19, 1936. Discussion of boiler feed water softening and of removal of iron, manganese, and dissolved gases. Softening by lime-soda, soda, sodium hydroxide, base exchange, and trisodium phosphate methods are dealt with. Inorganic salts of iron are removed by aëration, while sulfates and organic

salts of iron are precipitated with lime, or alum. Manganese permutite filters and bacteriological methods are also successful. Manganese is removed by means of lime, calcium aluminum sulfate, or manganese permutite filter. Specially prepared double filters can be used for simultaneous removal of iron and manganese. Chemical removal of oxygen from feed water and condensate is also discussed.—*R. E. Thompson.*

**Boiler Feed Water Treatment in the Palm Oil Factory.** C. D. V. Georgi. Malaya Agr. J., 23: 568–79, 1935. From Chem. Abst., 30: 2674, April 20, 1936. Hardness is practically non-existent in Malayan supplies. Most waters are acid (pH 3.0–6.8), often containing sulfuric acid in addition to such organic acids as humic. Some humic-acid-containing waters become alkaline upon boiling and thus require no treatment for boiler use. Removal of suspended matter by sedimentation, removal of oxygen, and treatment of permanently acid waters with sodium carbonate are recommended.—*R. E. Thompson.*

**Removal of Oil from Condensate for Use in Boilers.** A. C. Stutson. Power, 79: 582–4, 1935. From Chem. Abst., 30: 795, February 10, 1936. Clean water was obtained from condensate containing 30 p.p.m. emulsified oil by coagulation with ferric sulfate at pH 5.5 to 8.6 and filtration. Boilers are treated to maintain 20 to 30 grains per gallon of alkalinity, 30 p.p.m. phosphate, and 1000 to 17,000 p.p.m. sulfate.—*R. E. Thompson.*

## GROUND WATER AND WELLS

**Operating Distant Wells.** D. L. Erickson. Eng. News-Rec., 116: 765–6, May 28, 1936. Remote control of well pumping in water system of Lincoln, Nebraska, has not only demonstrated its effectiveness in simplifying operation, but has proved of unusual value during past winter when heavy snow prevented access to well field for extended period. Supply works include 7 wells, iron and manganese removal plant, and pumping station, located in Platte River Valley not far from Ashland, and 36-inch pipe line which conveys water 25 miles to Lincoln. Each well is equipped with electrically-driven deep-well turbine pump with capacity of 2 m.g.d. Treatment plant and pumping station have maximum capacity of 13.5 m.g.d. Distance from wells to pumping station varies from 2 to 4 miles. Operation is controlled from pumping station, deep-well pumps being governed by automatic telephone-type supervision. As routine, each well is visited once during each 8-hour shift. Well houses are heated by thermostatically-controlled electric strip heaters. Only 2 men and chemist are needed during day and one man on each of other 2 shifts.—*R. E. Thompson.*

**Supervision of Well Drilling a New State Function.** F. R. King. Wisconsin State Board of Health Bulletin, 5: 24, 6, October–December, 1935. First among the states to provide for both registration and regulation in well drilling, Wisconsin was to inaugurate this function January 1, 1936. New law provides that State Board of Health may establish minimum standards, rules, and regulations to be followed in well drilling. Chief aims of new procedure will be to insure proper location and construction of new wells.—*G. C. Houser.*

**New Water-Supply Method Used in England and Portugal.** Anon. The American City, **51**: 11, 43–45, (1936). The Ranney patented water collector, a development resulting from American experience in oil and water fields, has been successfully applied in augmenting the water supplies of London and of Lisbon. London installation consists of cast iron lined well, from which near the bottom, nine perforated steel screen pipes project horizontally and radially into the ground, extending to as much as 165 feet from well. Flow into well is controlled by valve on each pipe. Because of greater thickness of water-bearing sand, no radial pipes were used in Lisbon installation. For upper 55 feet, well is lined with reinforced concrete; for lower 50 feet, with circular screen sections grouped around a central steel and concrete casing. Advantages claimed are:—(1) exposure of a very large surface of water-bearing material; (2) selective removal of fine material from ground adjacent to screens; (3) independent control of, and back-washing facilities for, the separate sections, preventing incrustations and silting; and (4) fact that large diameter of well reduces drawdown and pumping costs.—*Arthur P. Miller.*

**Improving the Well Water Supply in Peiping, China.** P. K. Tao and C. S. Hsieh. Civil Eng., **6**: 6, 411 (1936). In Peiping, a city of one million inhabitants, only 15 percent use public water supply and remainder are dependent on wells. Typical well consists of a tube terminating in an open basin into which water rises under own pressure and from which water is drawn in willow buckets. Walls are of brick and plaster and not watertight, few wells have curbs, none covers and most are located within 50 ft. of latrines. A description is given of methods taken to improve wells after twice daily chlorination failed to help condition. Basin walls were plastered, top covered with concrete cover and hand pump installed discharging into covered tank. 30 wells have been improved at a cost of approx. $30 each.—*Martin E. Flentje.*

**Public Ground Water Supplies in Massachusetts.** Francis H. Kingsbury. Jour. New Eng. W. W. Assoc. **50**: 2, 149–96 (1936). 97.2 percent of inhabitants of Mass. were served by public water supplies in 1930 and 195 communities with population of 958,000 get supply wholly or in part from ground water. A table is given listing 159 communities and showing in tabular form the ground water supplies used by each, listing the information by decades and showing dates of new additions, changes, depth and character of wells, etc. Graphs are also shown by which the growth and decline of various types of wells and supplies can be traced, together with several pages of illustrations of the common types of ground water structures. Springs, dug wells, filter galleries, tubular and gravel packed wells have all been used in Mass. The gravel packed well is a natural development of the tubular well but the author points out this type of well should be considered a progressive member of the well family and not necessarily one capable of startling performance. Encroachment by salt water in areas near ocean is discussed. Ground water supplies must be protected and illustrations are given showing how increases in iron have in some cases been accompanied by increases in ammonia content. Regular tests should be made of these supplies to insure works remain safe, one of simplest tests being test for color. Ground waters are usually low in color and

change should be viewed with suspicion. In conclusion author suggests question should probably be raised whether or not there has been a measure of overconfidence concerning ground water supplies, and suggests since ground water is only naturally filtered water it should be protected from pollution as carefully as water artificially filtered.—*Martin E. Flentje.*

**Sanitary Investigation and Interpretation of Artificial Groundwater Enrichment.** FRANZ W. SCHÜBEL. Archiv für Hygiene und Bakteriologie, 116: 6, 321, September, 1936. History to date of a groundwater enrichment development is given. Since 1914, water supply of city of Bamberg has been derived from a gallery of wells in forest water works, driven in diluvial sand layers of 33 to 50 foot thickness, with underlying sandstone. Resulting depression of water table has been countered by infiltration of water from a brook catchment area almost coincident with the protected wooded district of the water works. Brook flow is extremely variable, depending on weather conditions and mostly ceasing during summer months. Infiltration water, which averages about 462,000 gallons per day, is cut off after heavy rains, on account of increased turbidity. It passes through sedimentation basin of 133,850 gallons capacity, equivalent to detention period of 5 to 8 hours, and along a ditch 623 feet in length to the infiltration basin of 105,600 to 205,900 gallons capacity with bottom area of 35,510 square feet, situated 492 feet from well gallery. Colouring with fluoresceine reveals detention time in the ground of at least 66 days, rate of travel varying from 18.4 to 1.47 feet per day with a gradient of 1¼ percent. Ratio of brook water to real ground water is as 1 to 1.5. In spite of varying temperature of brook water, complete equalization takes place, with yearly fluctuation of 1.9° near well gallery. Chemical and bacteriological tests show excellent purification of infiltration water.—*Manz.*

**Artificial Groundwater.** D. A. LANE. Eng. News-Rec., 116: 779–80, May 28, 1936. Artificial replenishment of ground water storage basins at Tujunga spreading grounds, gravel area in San Fernando Valley, has been successfully practised during past 5 years, safeguarding source that produces about 30 percent of Los Angeles domestic water supply. Location was selected so that water spread during winter months would be available following summer at North Hollywood pumping plant some distance down valley, which is fed from 18 wells 20 inches in diameter and averaging 400 feet deep, with total capacity of 100 second-feet. Plant functions strictly as emergency unit: if underground flow is not interrupted at that point, it is available few years later further down valley at points where diversion by galleries and pumps is more economical. Observations at key wells and at several hundred private wells have demonstrated effectiveness of project. Underground flow can be closely followed for several miles. Spreading grounds have area of 183 acres, of which 47-acre portion now in use represents capacity of 106 second-feet, i.e., as much as can be drawn from Owens River aqueduct without interfering with service to areas lower down. Spreading is carried out mainly in winter. Maximum quantity available has never equalled capacity of grounds, which is calculated at 30,000 acre-feet per season. Since 1931, total of 110,000 acre-feet has been applied to grounds. Area is divided into basins averaging 100 x 400 feet in size. Average

height of checks or dikes between basins is 4 feet and base width 15 feet, size being determined mainly by silt and soil which had to be removed from basin floor to expose best materials for spreading. Flow into each basin is through 6-inch pipe, valve provided being adjusted to maintain average water depth of 6 inches. Greater depths permit wave action, which endangers side walls. There is no overflow, nor are basins interconnected. Bottom of basins is cultivated with spring-tooth harrow drawn by small tractor. Small ridges thus formed are kept open by wave action and silt settles in valleys. Immediately after harrowing, rates of percolation may be as high as 10 feet or more per day, decreasing to 3 feet if operation is continued without interruption for long periods. Harrowing at intervals of about 10 days gives best results. Time required for drying and harrowing basin varies from 8 to 24 hours, depending on weather. Unit rates of percolation in acre-feet per day during past 4 seasons averaged 6.15, 6.92, 5.51, and 6.24. Low rate in 1933–4 was due to long period of operation with silty water.—*R. E. Thompson.*

## SUPPLY LINES

**Long Flume Service.** B. H. SKILLINGS. Eng. News-Rec., 116: 777–8, May 28, 1936. Gagings on 3-mile, old, wooden flume showed 13 percent loss through leakage and as annual maintenance costs had averaged $3,700 for 10 years it was decided to replace it with metal flume. Flume was originally constructed in 1870 and practically reconstructed in 1888; since when, it is believed, all sections have been replaced at least once. Flume box, 28 inches wide and 26 inches deep, was made of 1-inch redwood, mostly 14 inches wide. It was estimated that standard metal flume, 45.84 inches in diameter and costing $102,500, would have the desired capacity (10 m.g.d.). Several short sections of 29-inch pipe were replaced with 33-inch concrete pipe. Work was carried out in 3 successive years and has now been completed. Sinuous nature of route, partly along precipitous slopes, is indicated by fact that total curvature in 16,601 feet is 4,566 degrees. Creosoted timber substructure was used and entire length was covered with transverse boards. Notwithstanding that at many places timber sheds were built over flume, rock slides in January, 1935, following unusually heavy rain carried out 200-foot section. Flow measurements indicate delivery between 9.5 and 10 m.g.d.—*R. E. Thompson.*

**Twin 36-Inch Wood-Stave Pipes Moved Bodily Across Channel.** Eng. News-Rec., 116: 447–8, March 26, 1936. Brief description of construction of trunk sewer across Salmon Bay, Seattle. Crossing is about 1250 feet long, maximum water depth is 43 feet, and channel bottom is soft and uncertain and may shift slightly. Partly on this account, continuous wood-stave pipe was selected, life of permanently submerged wood being believed to be "almost indefinite." Twin pipes were placed, staves being machined from 3 x 6-inch selected clear fir stock to thickness of 2.5 inches. Galvanized steel bands, ¾ inch in diameter, encircle pipe at 10-inch intervals, under tension that beds them slightly into wood prior to swelling. After immersion, swelling increases initial stress in bands up to 18,000 pounds per square inch, or one-third their elastic limit. Lines were placed in trench 10 feet deep which was backfilled with 5 feet of pit-run gravel as protection against marine borers and dragging

anchors. Assembly was effected on bank and twin unit shoved across channel with cable and hoisting engine as work progressed, concrete weights being used to impart slight negative buoyancy. Crossing was completed in 60 days. Cost of pipe in place in trench was $16 per linear foot of 36-inch pipe.—*R. E. Thompson.*

**Novel High Pressure Grouting Methods Seal Fissures in Wet Tunnel.** C. R. RANKIN. Eng. News-Rec., 116: 661–3, May 7, 1936. In advancing east Potrero heading of San Jacinto tunnel of Colorado River aqueduct, under construction by Metropolitan Water District of Southern California, sudden water inflows in large quantity have been encountered, not only at face, but also from sides. Large pump capacity provided is ample to take care of water, but effective sealing of fissures through which water enters has been primary problem. This has been effected chiefly by cement grout applied under high pressure, in some cases with aid of sawdust at outset, working in successive zones with increasing pressures. In few extreme cases, timber bulkheads have been necessary to resist enormous pressure developed by full-face grouting. Details of 3 methods employed, advance grouting, rings of air-applied concrete, and grouting behind bulkheads, are given. Quick hardening cement was employed and 3 percent calcium chloride added to speed setting. Water pressures as high as 560 pounds per square inch have been encountered.—*R. E. Thompson.*

**The Greatest Long-Distance Water Supply System in Europe, Harz-Bremen.** F. COLLORIO. Gas- u. Wasserfach, 78: 470–83, 1935. From Chem. Abst., 29: 6983, October 20, 1935. Data given regarding construction of long steel pipe line. Interior was coated with bitumen compound by "flooding." Exterior was protected where necessity was indicated by soil survey. Mechanical joints were used instead of welding to avoid damage to interior coating. Water treatment employed includes use of chloramine, alum, and lime and filtration through manganese-containing sand filter to remove iron and manganese.—*R. E. Thompson.*

**Brook Diversion to Improve the Manchester, N. H., Water Supply.** PERCY A. SHAW. Jour. New Eng. W. W. Assoc., 50: 3, 291–98, (1936). Studies started in 1933 have resulted in changes to water supply of Manchester, N. H., two of which are described in this article. Supply is taken from Massabeic Lake, made up of twin ponds known as East Pond and West Pond. Because of city's growth to West Pond, a tributary stream to this pond—Spring Valley Brook, was diverted in 1935 to flow into East Pond. Article describes PWA project under which this was done. Provided employment for 175 men for 4 months, cost $55,000, city's share $9,300. Another tributary stream, Slager Brook, was diverted because of pollution. Slager Brook has watershed area of about 700 acres and a population of 568, and was diverted into Hogg Brook at cost of $65,000, of which $25,000 was for pipe and other materials.—*Martin E. Flentje.*